SOILS AND FOUNDATION
REPORT NO. 06-08

PROJECT PRA-BLRI 2P16
WALL SLIDE REPAIR, MP 364.72

BLUE RIDGE PARKWAY
BUNCOMBE COUNTY, NORTH CAROLINA

U.S. Department of Transportation
Federal Highway Administration
Eastern Federal Lands Highway Division
21400 Ridgetop Circle
Sterling, VA 20166

June 2008

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Note: Design changes subsequent to publication of this report and prior to the project's advertisement will be documented by a memo inserted after the title page.

GEOTECHNICAL
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INTRODUCTION

General

This report presents the results of our geotechnical subsurface investigations, design analyses, and summarizes our recommendations for wall slide repair in Project BLRI 2P16. The project site is located along the Blue Ridge Parkway at Milepost 364.72 in Buncombe County, North Carolina. The general site location is shown in Figure 1, "Site Location Map" presented in Appendix A.

Project Description

Project BLRI 2P16 is located in the Asheville District of the Blue Ridge Parkway. A recent collapse of a mortared stone retaining wall occurred at the above-referenced location. The wall slide is centered around an 18-inch diameter pipe culvert that has an outlet at the face of the wall. The collapsed section of the wall is approximately 54 ft long with a vertical scarp located at approximately 15 feet behind the collapsed wall backface. A vertical slide scarp is located at approximate distance of 13-ft beyond the back of the guardrail at the time of field investigations. The height of the wall at the collapsed section is estimated between 25 ft and 30 ft. The retaining wall supports a portion of the Blue Ridge Parkway embankment leading to Craggy Pinnacle Tunnel and Craggy Gardens Visitor Center and supports a 2-lane asphalt concrete (AC) paved road.

In addition to the collapsed section of the wall, there are signs that the adjacent sections of the wall are facing eminent failure. Shearing and bulging of the mortared stone facing was observed at these locations. The stone wall facing located immediately to the south of the collapse area has separated from the rock fill wall portion. Field measurements indicated that these adjacent sections extend a distance of 26 ft from the northern end of the collapsed wall and 28 ft from the southern end of the collapsed wall sections. The exposed backfill material behind the existing wall was observed to consist of silty sand soils and shot rock.

This project consists of re-building the collapsed section of the retaining wall and stabilizing the northern and southern adjacent unstable sections of the wall.

Existing Retaining Wall Description

Based on the observations provided in the field trip report (FTR) and during the subsurface field investigations, a 600± ft stone wall is located at the toe of the fill for the roadway embankment. The wall appears to consist of 4 to 6 ft wide shot rock and a mortared stone facing that was constructed against the shot rock. The thickness of the mortared stone face wall varies from 18 to 24 inches. The stone facing is constructed at a slight batter. The wall varies in height from 20 to 30 ft.

Regional Geology

The Geologic Map of North Carolina (1991) indicates that the project site is located in the Blue Ridge Physiographic Province and is underlain by rocks of the Ashe Metamorphic Suite and Tallulah Falls Formation. This geologic formation consists primarily of muscovite-biotite gneiss with a profile that consists of a layer of residual soil (micaceous silty sand and rock fragments) of varying thickness. The residual soil is underlain by weathered rock (fractured), that grades into un-weathered sound rock. Site geologic map is presented in Figure 2.

Soil Survey map of Buncombe County, NC (1954) indicates that surficial soil at the site belongs to the Stony rough land (Porters soil material). This formation is widely distributed on steep to very steep relief over the mountain section. It composes of rock outcrops and loose stone with some soil admixtures. Bedrock and stones are predominantly granite, gneiss, and schist. Surface runoff is very rapid, and some parts gully rapidly when cleared of forest. The soil survey map is shown in Figure 3.

PROCEDURES AND RESULTS

Soil Borings and Rock Coring

A field investigation was conducted by the Eastern Federal Lands Highway Division (EFLHD) Geotechnical Subsurface Investigation Team between January 8th and 11th, 2008. The field investigation consisted of drilling/coring four (4) Borings (B-1 through B-4). Borings B-1, B-2, and B-3 were drilled in the eastern shoulder of the Parkway, while Boring B-4 was drilled in the western side across from the center of the collapsed wall section. Boring B-1 was drilled to the south of the collapsed area, Boring B-2 was drilled 10 ft to the north of the collapsed pipe culvert, and Boring B-3 was drilled to the north of the collapsed area.

Rock coring was performed in all of the borings. Once auger refusal was encountered, rock coring was performed at 2 to 3 ft offset locations. Rock coring was performed using rotary drilling techniques and samples were retrieved using an NQ core barrel and wireline. Rock core samples were preserved in wooden boxes for laboratory testing.

Borings were advanced to refusal depths using hollow-stem augers by CME 750 ATV-mounted rotary drill rig. All boreholes were backfilled with auger cuttings upon completion. Boreholes were laid out by EFLHD field personnel by measuring distances from mapped landmarks. Borings were graphically depicted on the Boring Location Plan and Subsurface Profile sheets in Appendix B. Boring Logs are presented in Appendix C.

Sampling

Sampling of materials beneath the tip of the hollow stem augers was performed in all borings. Standard Penetration Testing was performed using a 2¼-in. (outside diameter) split-spoon sampler in accordance with AASHTO 7200-87 and AASHTO T206-87. SPT soil samples were typically recovered at 2.5 ft intervals by driving the split-spoon sampler a distance of 24-in. into the undisturbed soil under the impact of a 140 lb. automatic hammer free-falling 30 inches. The number of hammer blows required to advance the split-spoon sampler the middle foot of the 24-in. sample interval is designated as the “Standard Penetration Resistance” or N-value. The number of blows required to advance the sampler through each 6-in. interval was recorded on field boring logs. Upon completion of the SPT’s, the sampler was removed from the ground and sample recovery measurements were made and recorded for each sampling event. A field description by color and texture was made for each recovered sample. Representative portions of split-spoon samples were preserved in glass jars. Water levels, if present, were measured in the boring logs at the time and under the conditions stated on boring logs.

The sampling sequence for soil borings is summarized on the Boring Logs presented in Appendix C.

Geophysical Survey

Eastern Federal Lands Highway Division (EFLHD) conducted a seismic refraction survey at the project site on January 7th, 2008. Two (2) seismic refraction survey lines were performed at the site using a Smartseis S24 System with 24 channels. Seismic refraction survey lines were performed along the eastern (Line 0001) and western (line 001) sides of the road and covered the collapsed area as well as the adjacent sections. Geophones were spaced at 10 feet intervals, and total geophone array measured 230 ft. Shots were typically taken at the first and last sensors and at approximately 40 ft intervals in between. Shots were also taken at 5-ft, 20-ft, and 50 ft offsets from each end of the array, for a total of 13 shots per line. Shots were produced with a sledge hammer on a striker plate. Blackhawk – a Division of Zapata Engineering of Golden, Colorado, processed the geophysical data collected by EFLHD. Their report, dated February 14th, 2008, is included in Appendix D.

The approximate location of the seismic refraction lines and soil borings is shown in Figure 4 and soil profiles, obtained from the soil borings, are included in Figures 5A and 5B, Appendix B.

Field Tests and Measurements

The EFLHD geotechnical crew performed field tests and took measurements during the course of the subsurface exploration.

A field description by color and texture was made for each recovered soil sample. Percent core recovery (CR) and rock quality designation (RQD) were determined for each core run to provide a quantitative basis for evaluation of the conditions of the rock.

The seismic refraction survey and boring locations were determined through a combination of GPS coordinates and field measurements with a tape measure. Seismic refraction survey line elevations were determined by overlaying the refraction lines on the topographic plan, and checking with approximate field measurements.

Data Summary

The results of field tests and measurements were recorded on the driller's logs and appropriate data sheets in the field. These data sheets and logs contain information concerning the boring methods; samples attempted and recovered; indications of the presence of various material such as gravel, pebbles, organic matter, etc.; and observations of groundwater. They also contain interpretations by the exploration foreman of the subsurface conditions based on the performance of the equipment and cuttings brought to the surface by the drilling tools. Therefore, the field data represents both factual and interpretative information.

The boring logs in Appendix C of this report represent a compilation of field laboratory data and description of the soil samples by a geotechnical engineer. These records occasionally do not include all data recorded on driller's logs and field data sheets, but do include all information considered relevant to the design and preparation of this report.

Groundwater level readings were made in the boreholes at the times and under the conditions stated on the boring logs. However, fluctuations in groundwater level due to seasonal variations, rainfall, temperature, and other factors not evident at the time measurements were made should be expected.

Laboratory Testing

A laboratory testing program was conducted on representative rock samples recovered during the subsurface explorations. The primary purpose of the testing program was to aid in evaluation of the engineering properties of rock present at the site. Samples were tested for unconfined compressive strength (ASTM 2938). All tests were conducted in accordance with applicable ASTM/AASHTO standard test methods.

The results of the laboratory testing program are presented in Appendix E and summarized below in Table 1.

Table 1. Summary of Laboratory Test Results

Boring No.	Run No.	Sample Depth (ft)	Unconfined Compressive Strength (psi)	Rock Description
B-1	5	23.6-28.6	4,690	Gray Micaceous Schist
B-2	2	18.7-23.7	5,920	Gray Micaceous Schist
B-3	4	28.5-33.5	3,040	Gray Micaceous Schist

Findings

Soil Borings

Descriptions of the subsurface conditions encountered in the soil borings are provided below. The stratification lines designating the interfaces between soil types on the boring logs in Appendix C represent approximate boundaries. The transition between materials may be gradual. It should be noted that one or more of the units may be absent at specific locations.

TOPSOIL **–** Topsoil encountered in the soil borings ranged in thickness from 2 to 4 inches.

FILL - The fill materials encountered in the borings consisted primarily of brown, micaceous silty sand with various amounts of clay and rock fragments. N-values recorded within this stratum ranged between 2 to 20 blows per foot (bpf); indicating very loose to medium dense relative densities. The stratum depths encountered in the borings ranged from 3 ft in Boring B-4 to 13.7 ft in Boring B-2.

BEDROCK - Light gray to gray, slightly weathered to moderately weathered, fine to medium textured, moderately hard to hard, with foliation angles of 30 to 90 degrees micaceous schist was encountered in all borings. Rock quality designation (RQD) values measured from the retrieved rock cores varied from 0% to 95% indicating highly weathered to competent rock.

GROUNDWATER - Groundwater was encountered at a depth of 10 ft below existing site grades in Boring B-3. It should be noted that auger refusal was encountered at shallow depths in the remaining three borings. Because water was introduced during the rock coring process, no water measurements were possible in the remaining borings. Fluctuations in the groundwater conditions due to seasonal weather changes should be anticipated.

Summary of our field measurements are listed below:

Table 2. Summary of Field Measurements

Boring No.	Depth to Auger Refusal (ft)	Boring Termination Depth (ft)
B-1	4.6	28.6
B-2	13.7	23.7
B-3	3.5	33.5
B-4	3.0	19.0

Seismic Refraction Survey

The purpose of performing the seismic refraction survey was to map subsurface material, and identify depth to bedrock. The data obtained from the seismic analysis appears to generally agree with the results of our drilling operations.

DESIGN ANALYSIS AND CONCLUSIONS

Design Alternatives

A. Collapsed Section

Several alternative retaining wall types were considered for the re-construction of the collapsed wall section. Decision criteria used included: initial cost, constructability, area of disturbance required for construction, and proven design. Retaining wall types considered included:

1. Gabion basket wall.
2. Reinforced concrete gravity wall.
3. Reinforced concrete cantilevered wall.
4. Prefabricated modular block wall.
5. Mechanically stabilized earth (MSE) wall (Geosynthetically stabilized earth (GSE)).
6. Cantilivered soldier pile wall with concrete lagging.
7. Anchored soldier pile wall with concrete lagging.

We understand that a mortared stone face is planned for all of the above alternatives in order to restore existing site aesthetic conditions.

A brief description of each type of proposed retaining walls is presented below:

<u>Gabion Basket Wall</u>

These walls consist of wiremesh baskets that are filled with stone and placed on top of each other to form a self draining retaining wall. Gabion walls are essentially gravity type walls and would require considerable thickness in order to provide adequate stability. They are labor intensive and will require a considerable volume of imported rock to fill the baskets. In addition, the steel baskets are prone to corrosion in such atmosphere, reducing the service life of the retaining wall. Recently EFLHD has designed and constructed a geogrid reinforced type of gabion walls that can be considered at this site.

<u>Reinforced Concrete Gravity Wall</u>

Because of the measured height of the wall (25 to 30 ft high walls), the gravity retaining wall is likely to require considerable thickness, steel reinforcement and wide footings in order to provide adequate support and resistance to the lateral earth pressure and sliding. The wall also requires a properly designed and constructed drainage system to avoid the build-up of hydrostatic water pressure behind the wall. In addition, Foundations may need to be supported on a system of drilled shafts socketed into bedrock to provide additional stability and resistance against the lateral earth pressure and sliding. The cost associated with construction of this type of wall will be significantly higher than other alternatives.

Prefabricated Modular Block Wall

These walls consist of prefabricated units that are delivered to the site and assembled together to form self draining retaining walls. These walls are generally expensive and require high degree of quality control in the field to properly assemble the prefabricated units. Heavy machinery may be required during the construction process. Drilled shafts may also be required to provide additional foundation support and resistance to lateral earth pressure and sliding. The retaining wall is backfilled with granular selected fill.

Based on the above-mentioned evaluation, it is our opinion that the following two options represent the most economic and applicable alternatives for this site. These options are discussed briefly below:

Mechanically Stabilized Earth (MSE) Wall

These walls are constructed using common construction materials, and conventional equipment and techniques, providing economical and durable walls. MSE walls are self draining, cost effective, and tolerate larger differential settlements compared to other wall alternatives. MSE walls require the soil behind the wall face, through its entire width, to have proper drainage and compaction with sufficient reinforcement elements to develop pullout resistance behind the critical failure plan.

Assuming the height of the new wall is in the order of 25 to 30 ft, it is likely that the geogrid reinforcement elements will extend a distance of 25 ft horizontally in order to provide the appropriate pull-out resistance. Consequently, construction of the wall might require some rock excavation in order to achieve geogrid embedded length. However, because of the encountered competent rock behind the wall, a much shorter geogrid reinforcement embedment length would be required. A schematic diagram of this alternative is shown in Figure 6.

Analysis for the proposed MSE wall was performed in accordance with the design concepts and procedures presented in FHWA Publication No. FHWA-FLP-94-006 (1994), “Retaining Walls Design Guide” and FHWA SA-96-071 (1997), “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines”. Design computations of the wall were performed using the Federal Highway Administration’s ReSSA (2.0) Computer program.

The wall was designed for a maximum height of 28 feet and a 1:10 (H:V) batter of the stone facing. Granular fill within the reinforced zone should meet the requirements of FP-03, Sec.704.10. This fill should be naturally occurring, non- plastic sand classifying as A-2-4 (0) or coarser with less than 25% passing a #200 sieve and Plasticity Index (PI) of less than 6. Based on the results of our geotechnical study, we anticipate that the reinforced soil mass will need to be constructed using off-site borrow. The soil properties used in our computer modeling are summarized in Table 3 below:

Table 3. Soil Parameters for the Proposed MSE Wall

Soil Type	Unit Weight, γ (pcf)	Friction Angle, φ (deg)	Cohesion, c (psf)
Embankment	130	32	0
Retaining Wall (Fill)	130	34	0
Retained Soil (Rock)	145	40	1,200

Our analysis yielded a minimum factor of safety of 1.41 for global stability.
The results of our computer analysis are included in Appendix F.

Anchored Soldier pile (Steel H-pile) and lagging wall

This alternative is shown schematically in Figure 7 and consists primarily of a system of steel H-piles and timber (or concrete) lagging. Typically, drilled shafts 2 ft to 3 ft in diameter socketed into the underlying bedrock are utilized to provide foundation and lateral support for the H-piles. Rock anchors will be required to control horizontal movements and provide support against lateral earth pressure. Select granular fill materials should be placed and compacted behind the wall to eliminate the build-up of hydrostatic pressures and reduce future settlements. We estimate that the thickness of the granular backfill behind the wall will be in the order of 6 ft, which is essentially the same thickness of the collapsed wall.

This approach would require rock coring, both horizontally and vertically, and the installation of rock anchors. However, the volume of selected imported soils and rock excavations/ blasting will be reduced because the rock anchors will control the horizontal movement of the wall and the granular backfill will extend a distance of 6 ft instead of 25 ft for the MSE option.

Stabilization of Wall Slide Adjacent Sections

As mentioned earlier, the wall sections located immediately to the north and south of the failed wall exhibited signs of distress and bulging. Portions of the bulging wall sections may have to be reconstructed in order to properly achieve vertical and horizontal alignment. The remaining sections can be stabilized by installing rock anchors with steel plate anchor heads. A schematic diagram of the proposed anchor stabilization system is shown in Figure 6.

Based on our design analysis, the proposed walls repair consists of installing rock anchors at 8-foot by 8-foot grid pattern. Anchors were designed for 35 kips tension capacity. The design of rock anchors is described as follows.

Rock Anchors

The selected design alternative consists of installing rock anchors through the existing wall, weathered rock, and into bedrock (through the failure plane). The rock anchors were designed using principles for ground anchors as presented in FHWA's Geotechnical Engineering Circular No. 4 (1999) – "Ground Anchors and Anchored Systems." An allowable rock-grout bond stress of 450 psi was calculated based on laboratory rock strength data. A minimum bond length of 10 feet was calculated.

Rock anchor data used in our analysis are summarized below:

- Anchor tensile working load : 35,000 lbs
- Allowable rock-grout-bond strength : 45 psi
- Anchor Bond Length : 10 feet
- Anchor Slope : 15 degrees (from horizontal)

Analyses results indicate that the proposed rock anchors will provide adequate resistance for horizontal wall movement. Refer to Appendix G for details of anchor design calculations.

RECOMMENDATIONS

Retaining Wall

The results of the design analyses are summarized below in Table 4. The computer generated output files from ReSSA (2.0) program are provided in Appendix F. These output files provide the soil input parameters, the slide section geometry analyzed, reinforcement spacing, type, strength and interaction parameters, the calculated factors of safety for the stability criteria presented above, and plots of the calculated critical failure planes.

Table 4. Retaining Wall Configuration (Sta. 11+25)

Wall Height (ft)	28
Minimum Reinforcement Embedded Length (ft)	7 in the base zone 14 in the middle zone 27 in the upper zone
Geogrid Design Tensile Strength (lb/ft)	4,500
Maximum Geogrid vertical Spacing (ft)	1.5
Minimum Allowable Foundation	4,000

Bearing Capacity (psf)*	
Face Batter	1(H):10(V)

*: Wall foundations will be anchored to bedrock using rock dowels in order to improve slideing factor of safety.

It is recommended to install a minimum geogrid embedment length of 15 ft for the roadway embankment portion constructed above the top of the retaining wall.

Geogrid reinforcement should be installed in accordance with Section 714 of the FP-03 Specifications. Space geogrid layers at a maximum vertical spacing of 1.5 ft.

Rock Anchors

Provide anchors that meet the following requirements:

Anchor type: Bar Tendon

Anchor diameter: 1 3/8 inch

Anchor grade: 160 (160 ksi ultimate tensile strength)

Anchor minimum bond length: 10 ft

Anchor minimum unbonded length beyond the critical failure zone: 5 ft

Anchor slope: 15° from horizontal

Corrosion protection: Epoxy coated or galvanized, and completely surrounded by cement grout.

Bearing plates: 12” by 12”, maximum

Spacing: 8 ft maximum center to center

Anchor design load capacity: 35,000 lb

CONSTRUCTION CONSIDERATIONS

Ground and Surface Water Management: Control of storm water or seepage water flowing into open excavations and through the cracks between the rocks is necessary in order to maintain dry conditions during construction. The contractor should control the flow of surface water and seepage water into excavations at all times. Storm water collection and control during construction may be performed using collection trenches and sumps, if accumulation does occur.

Backfill Material: Backfill material for the slope repaire should consist of AASHTO A-2-4 material or better. The maximum dimension of coarse aggregate used for backfill material should not exceed 4 inches. Backfill material should be placed and compacted in lifts not to exceed 12 inches, per Section 204 of the FP-03 Specifications. The portion of the on-site excavated material that meets the unclassified borrow specification may be used as backfill material.

Backfill material consisting of selected granular fill is preferred and recommended. Selected granular fill will expedite the construction process since it require minimum amount of compaction and can achieve the required density in wet weather conditions.

DISCLAIMER/LIMITATIONS CLAUSE

The subsurface explorations and test described in the section Procedures and Results have been conducted in accordance with standard practices and procedures (except as specifically noted). The results of these exploration and test represent conditions at the specific locations indicated. Subsurface conditions between these locations may vary. The Analysis and Conclusions sections and the Recommendations section of this report include interpretations and recommendations developed by the Government in the process of preparing the design. These interpretations are not intended as a substitute for the personal investigation, independent interpretation, and judgment of the Contractor.

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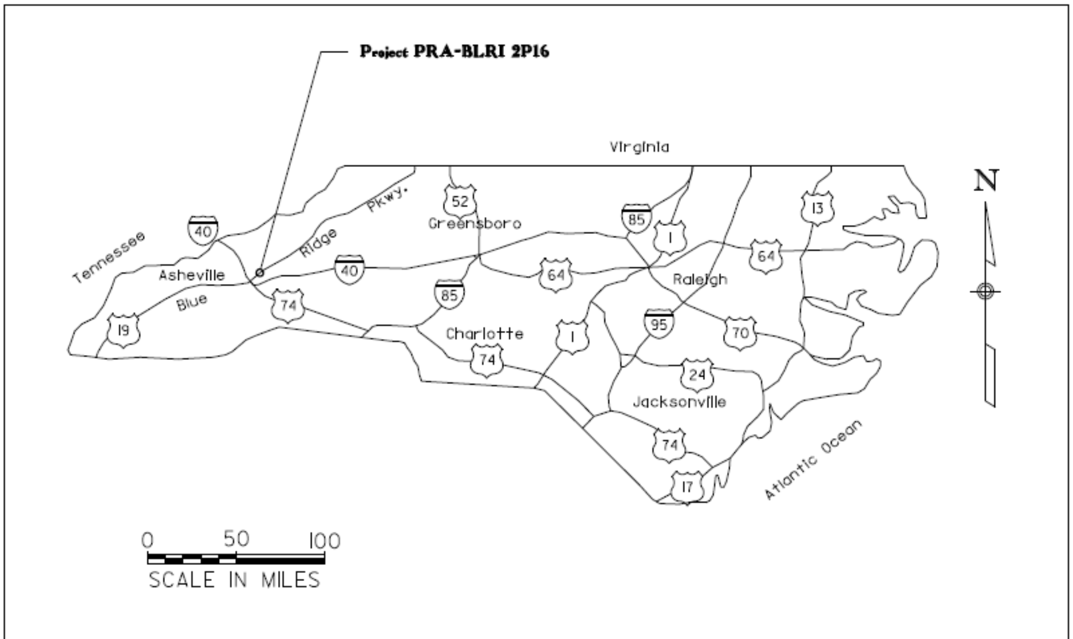
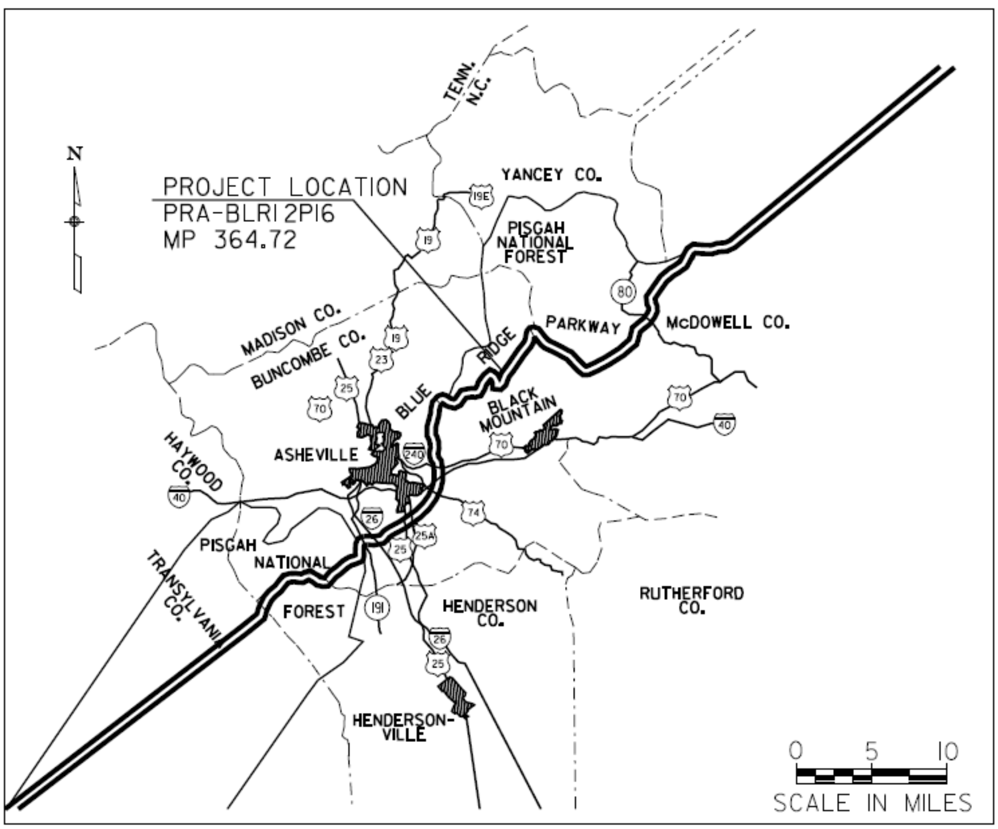
APPENDIX A
Figures

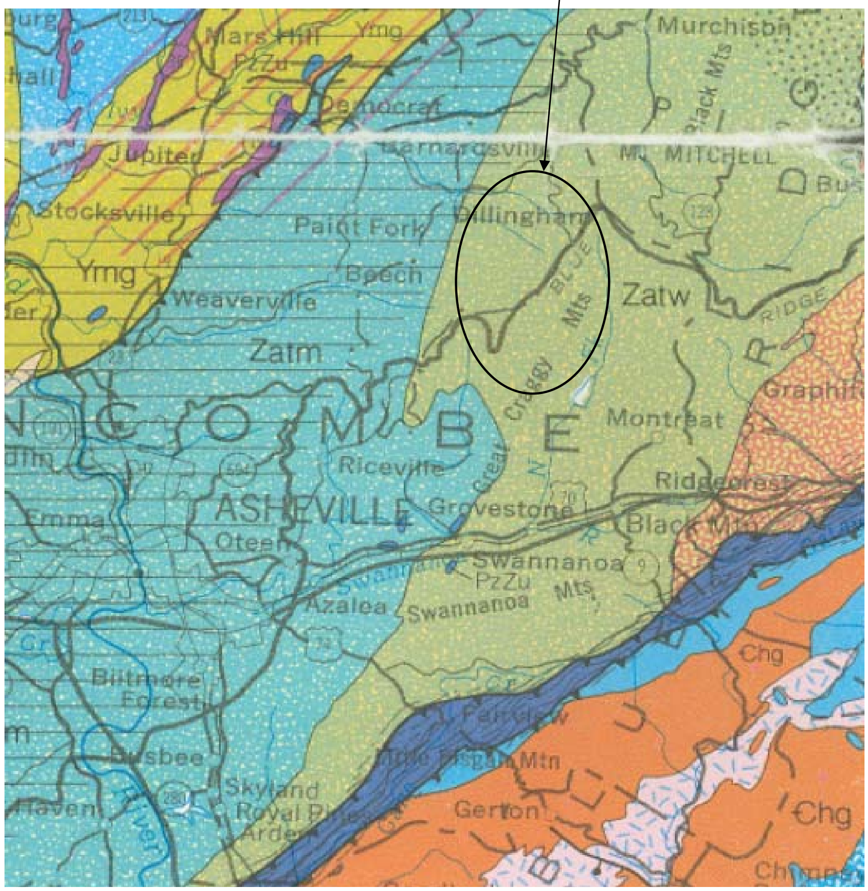


NORTH CAROLINA KEY MAP



U.S. DEPARTMENT OF TRANSPORTATION FEDERAL LANDS HIGHWAY DIVISION EASTERN FEDERAL LANDS HIGHWAY DIVISION STERLING, VIRGINIA	**SITE LOCATION MAP** BLUE RIDGE PARKWAY SLIDE AT MP364.72	REG	STATE	PROJECT	SHEET NO.	TOTAL SHEETS
		NE	NC	PRA - BLRI 2P16	1	3

Project PRA-BLRI 2P16



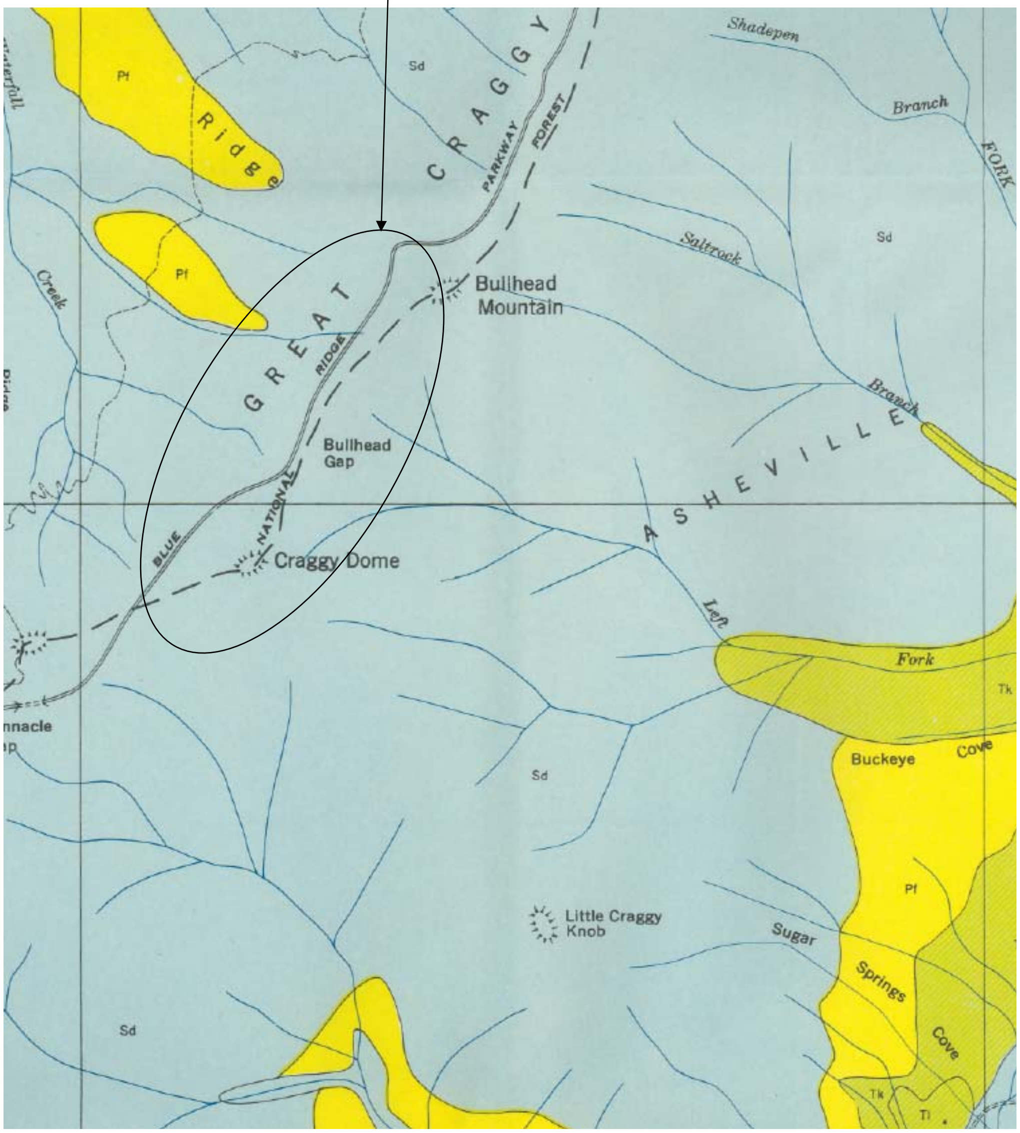
ASHE METAMORPHIC SUITE AND TALLULAH FALLS FORMATION

Zatw

Metagraywacke-Foliated to massive, locally conglomeratic, inter-layered and gradational with mica schist, muscovite-biotite gneiss, and rare graphitic schist.

U.S. DEPARTMENT OF TRANSPORTATION FEDERAL LANDS HIGHWAY DIVISION EASTERN FEDERAL LANDS HIGHWAY DIVISION STERLING, VIRGINIA	**GEOLOGIC MAP** BLUE RIDGE PARKWAY SLIDE AT MP364.72	REG	STATE	PROJECT	SHEET NO.	TOTAL SHEETS
		NE	NC	PRA-BLRI 2P16	2	3

Project PRA-BLRI 2P16



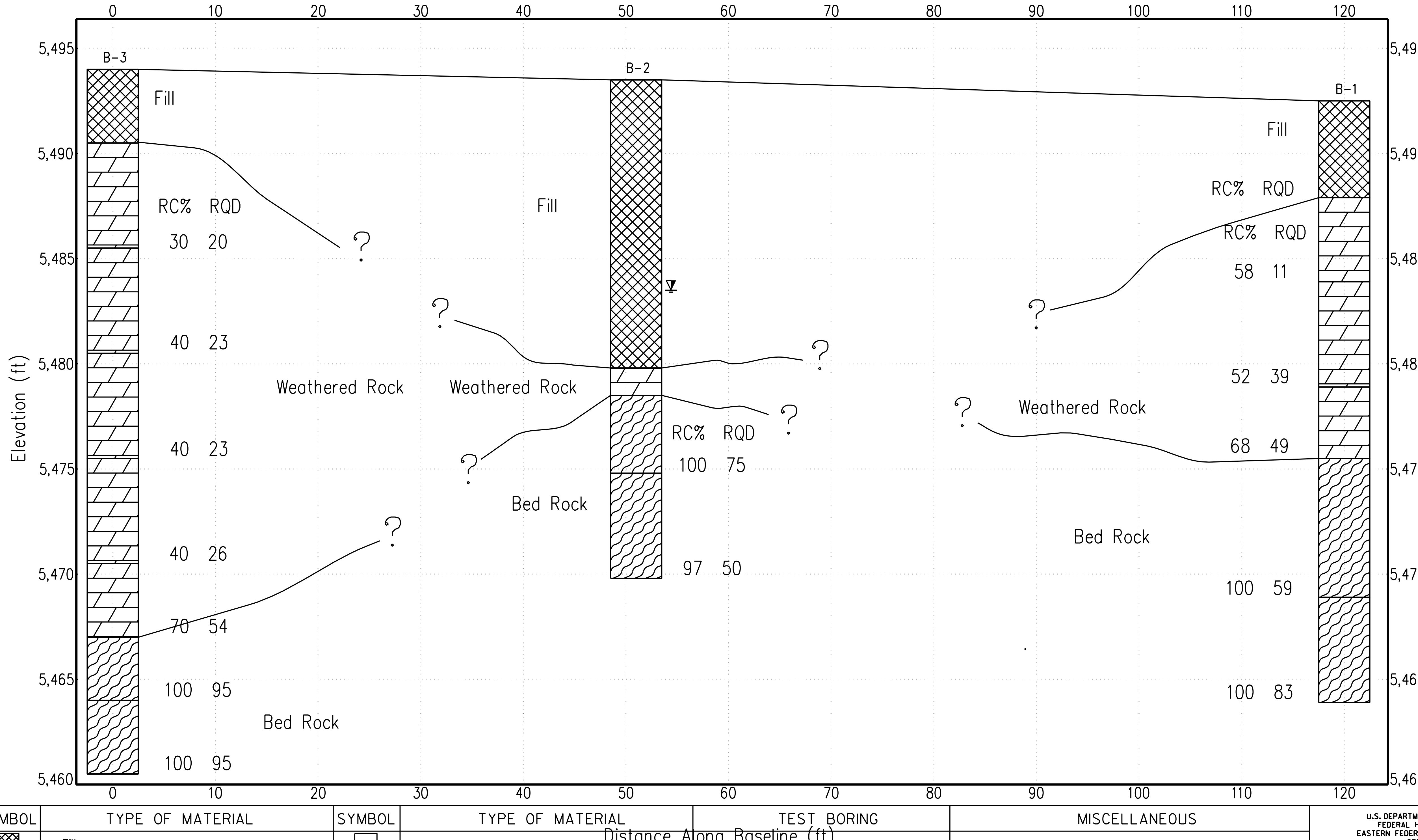
Stony rough land

(Porters soil material)

U.S. DEPARTMENT OF TRANSPORTATION FEDERAL LANDS HIGHWAY DIVISION EASTERN FEDERAL LANDS HIGHWAY DIVISION STERLING, VIRGINIA	**SOIL SURVEY MAP** BLUE RIDGE PARKWAY SLIDE AT MP364.72	REG	STATE	PROJECT	SHEET NO.	TOTAL SHEETS
		NE	NC	PRA-BLRI 2P16	3	3

APPENDIX B
Exploration Location Plan & Subsurface Profiles

REG	STATE	PROJECT	SHEET NO.	TOTAL SHEETS
NE	NC	BLRI.2P16	5A	—



SYMBOL	TYPE OF MATERIAL	SYMBOL	TYPE OF MATERIAL
	Fill		
	Decomposed Rock		
	Bed Rock		

TEST BORING

BORING NUMBER B-N

N BLOWS/12" (SPT)

J-N JAR SAMPLE NO.

WATER LEVEL (WL) (24 HOURS)

WATER LEVEL (WL) (TIME OF DRILLING)

CR% N N

RQD N N

DEPTH MARKS

BHT OR BHR

MISCELLANEOUS

1. SPT - STANDARD PENETRATION TEST - AASHTO T206-74
2. R - REFUSAL, SPT 100 BLOWS/12"
3. CR% - PERCENT OF RECOVERY
4. RQD - ROCK QUALITY DESIGNATION
5. BHT - BORE HOLE TERMINATED
6. BHR - BORE HOLE REFUSAL
7. GEOPHYSICAL TEST SITE: SEISMIC S RESISTIVITY R

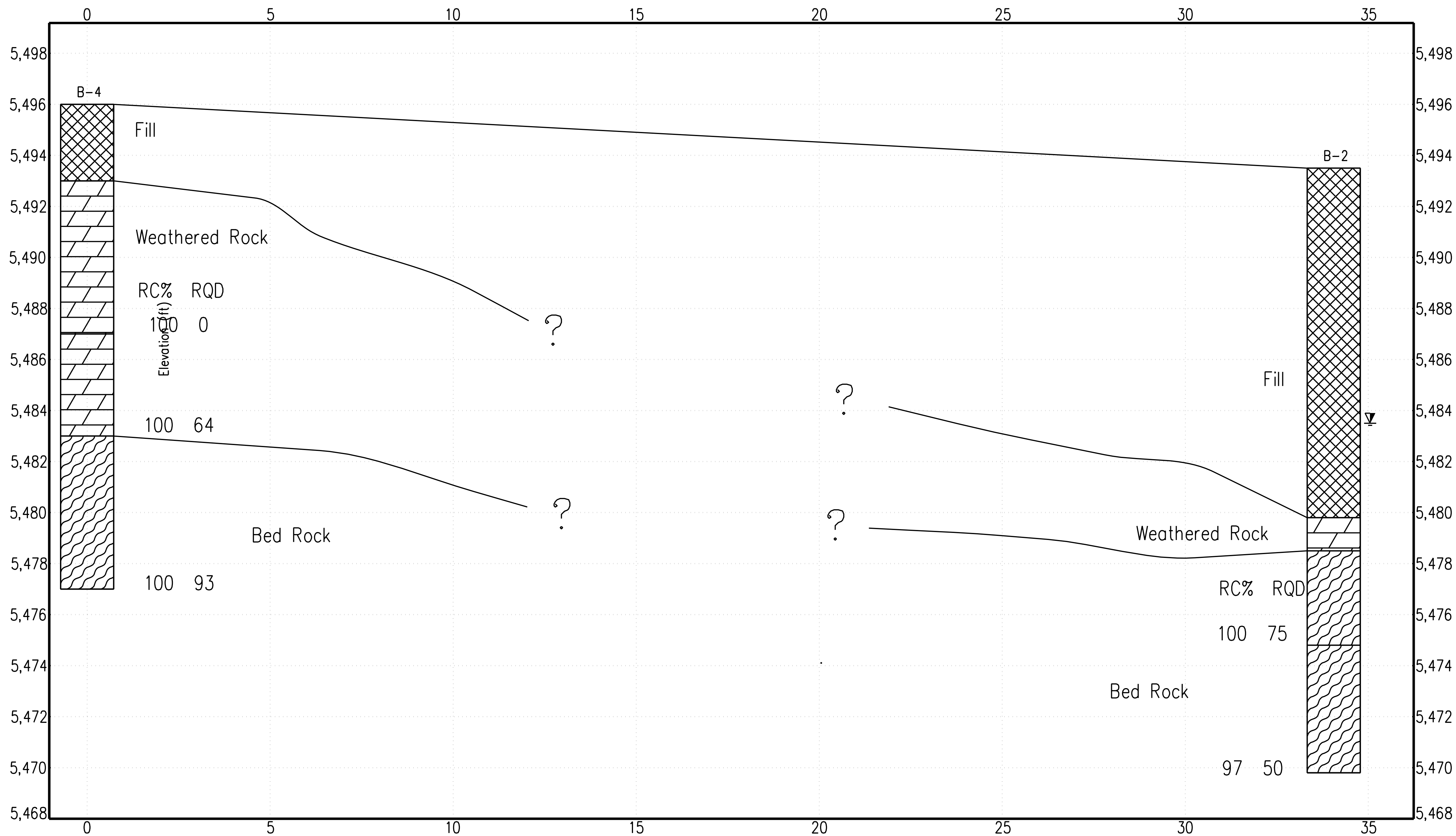
SCALE

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
EASTERN FEDERAL LANDS HIGHWAY DIVISION
STERLING, VIRGINIA

BLRI 2P16-Wall Slide at MP 364.72

Blue Ridge Parkway
Buncombe County, North Carolina

REG	STATE	PROJECT	SHEET NO.	TOTAL SHEETS
NE	NC	BLRI 2P16	58	—



SYMBOL	TYPE OF MATERIAL	SYMBOL	TYPE OF MATERIAL
▨	Fill	☐	
▨	Decomposed Rock	☐	
▨	Bed Rock	☐	
☐		☐	
☐		☐	
☐		☐	
☐		☐	

TEST BORING

BORING NUMBER B-N
WATER LEVEL (WL) (24 HOURS)
WATER LEVEL (WL) (TIME OF DRILLING)
DEPTH MARKS
BHT OR BHR
N BLOWS/12" (SPT)
J-N JAR SAMPLE NO.
CR%
ROD
N
N
N
N

MISCELLANEOUS

1. SPT - STANDARD PENETRATION TEST - AASHTO T206-74
2. R - REFUSAL, SPT 100 BLOWS/12"
3. CR% - PERCENT OF RECOVERY
4. ROD - ROCK QUALITY DESIGNATION
5. BHT - BORE HOLE TERMINATED
6. BHR - BORE HOLE REFUSAL
7. GEOPHYSICAL TEST SITE: SEISMIC S RESISTIVITY R

SCALE

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
EASTERN FEDERAL LANDS HIGHWAY DIVISION
STERLING, VIRGINIA

BLRI 2P16-Wall Slide at MP 364.72

Blue Ridge Parkway
Buncombe County, North Carolina

$$$$DGN$$ $TIME$ $$$$DATE$$$$

APPENDIX C
Boring Logs

SOIL BORING GENERAL NOTES

Drilling and Sampling Symbols

SS: Split Spoon - 1 3/8" I.D., 2" O.D., except where noted
ST: Shelby Tube - 2" O.D., except where noted
PA: Power Auger Sample

Water levels indicated on the boring logs are the levels measured in the boring at the times indicated. In pervious soils, the indicated elevations are considered reliable ground water levels. In impervious soils, the accurate determination of ground water elevations is not possible, even after several days, and additional evidence on ground water elevations must be sought.

VISUAL METHODS FOR SOILS CLASSIFICATION

Component	Distinguishing Features
Boulders	Larger than 12" (300 mm)
Cobbles	3" to 12" (75 mm to 12 mm)
Gravel	Larger than No. 4 sieve and smaller than a 3" sieve, described with any of the following terms (or any combination):
Coarse	3" to 3/4" (75 mm to 19 mm) sieve
Medium	3/4" to 3/8" (19 mm to 9.5 mm) sieve
Fine	3/8" to No. 4 (9.5 mm to 4.75 mm) sieve
Sand	The finest sand grains are just visible to the naked eye, while the largest would pass a No. 4 (4.75mm) sieve (pinhead size). Described with any of the following terms (or any combination):
Coarse	No. 4 to No. 10 (4.75 mm to 2.0 mm) sieve
Medium	No. 10 to No. 40 (2.0 mm to 0.42 mm) sieve
Fine	No. 40 to No. 200 (0.42 mm to 0.075 mm) sieve
Silt	1. Lumps are easily crumbled when are-dried. 2. Feels gritty between the teeth. 3. A moist pat when shaken in the palm of the hand will appear shiny and wet. When squeezed it will appear dry and dull.
Clay	1. Lumps are comparatively hard when air-dried. 2. Threads (1/8" diameter) of considerable length will support their own weight when held by one end. 3. A moist pat will appear the same whether shaken in the palm of the hand or squeezed.

Order of Description

1. Soil Density (or consistency) – see table below
2. Color
3. Major Grain Size – Composes more than 50% of the sample
4. Modifying Term – "and" : 40% to 50% of the minor grain size
   "some" : 30% to 40%
   "little" : 10% to 30%
   "trace" : 10% or less
5. Minor Grain Size(s)
6. Other (plasticity, etc.)

7. Moisture Content (by field test) – "dry" : Absence of moisture, dusty, dry to the touch
"moist" : Damp but no visible water
"wet" : Visible free water, usually soil is below water table
8. General Classification – Fill, Residual Soil, Weathered Rock

SOIL DENSITY (OR CONSISTENCY) TABLE			
Coarse-Grained Soil (Gravel, Sand)		**Fine-Grained Soil (Clay, Silt)**	
Apparent Density	SPT (# blows / ft)	Consistency	SPT (# blows / ft)
Very loose	0-4	Very soft	0-2
Loose	5-10	Soft	3-4
Medium dense	11-30	Medium stiff	5-8
Dense	31-50	Stiff	9-15
Very dense	>50	Very stiff	16-30
		Hard	>30

Examples:

1. Dense to very dense, brown to light brown, **SILTY SAND**, some gravel [A-7-6(10)]
(Moist)

-FILL-

Criteria for Describing Soil Structure

Description	Criteria
Bed	A sedimentary layer bounded by depositional surfaces.
Blocky	A characteristic in which cohesive soil can be broken down into small angular lumps which resist further breakdown.
Bonded	Attached or adhering.
Fissured	Broken along definite planes of fracture.
Foliated	Planar arrangement of textural or structural features.
Frequent	More than one per foot of thickness.
Homogeneous	Same color and appearance throughout.
Interbedded	Alternating soil layers of different composition.
Laminae	A very thin cohesive layer.
Layer	A general term for material lying essentially parallel to the surfaces against which it was formed.
Lens	A lenticular deposit, larger than a pocket.
Occasional	One or less per foot of thickness.
Parting	A very thin granular layer.
Pocket	Small erratic deposits less than 12" in thickness.
Seam	A thin layer separating two distinctive layers of different composition or greater magnitude.
Stratified	Alternating layers of varying material or color.
Stratum	A stratigraphic unit.
Varve	A cyclic sedimentary couplet consisting of a coarser and a finer layer representing the variation in depositional energy resulting from the annual freeze-thaw cycle typically found in glaciolacustrine environments.

ROCK CORING GENERAL NOTES

Depth and Elevation: Use large marks as 1' (300mm) increments. Record proper elevations.

Core: Draw sketch of core breaks as it is oriented in the core box (align all core breaks so they fit together properly before drawing sketch). Starting at the top of core, measure each piece of core down its centerline to 1/100 of a foot. Record this measurement along the left side of the core sketch at the break.

VISUAL METHODS FOR ROCK IDENTIFICATION

Description:

1. Draw a heavy line through description at depth to which core run penetrated.
2. Describe the rock type.
3. Note the condition of the core break on the right side of the core sketch
   Mud seam (MS); Sand seam (SS); Weathered surface (WS); Fresh break (FB)
4. Record coring time in minutes.
5. Record to nearest 1/100 foot the core recovered (after alignment in core box). Discard any debris at top of core, which obviously fill into the core hole.
6. Calculate per cent core recovery and record: $CR = \dfrac{\text{feet of core recovered}}{\text{feet cored}}$
7. Rock Quality Designation (RQD)
   $$(RQD) = \frac{\Sigma[\text{Lengths of all pieces of the core} \geq 4\text{" (100mm)}]}{\text{Total length of core run}} \times 100$$

Hardness:

Very Soft (VS)	Can be deformed or crumbled by hand;
Soft (S)	Can be scratched with a fingernail
Moderately Hard (MH)	Can be scratched easily with a knife;
Hard (H)	Can be scratched with difficulty with a knife;
Very hard (VH)	Cannot be scratched with a knife

Color: Wet the rock with water and describe the color including the color of any unusual or reoccurring markings on the core (i.e. light green with dark green bands, foliation lines).

Soundness: Use the proper number 1 through 4

1.	Weathered	RQD = 0% to 25%
2.	Highly jointed to Jointed	RQD = 25% to 50%
3.	Jointed to Relatively sound	RQD = 50% to 75%
4.	Relatively sound to Sound	RQD = 75% to 100%

Main Rock Formation Name

Texture

Very Fine (VF),
Fine (F),
Medium (M),
Course (C)

Modifying Term

"and"	40% to 50% of the core run
"some"	30% to 40%
"little"	10% to 30%
"trace"	10% or less

Minor Rock Type(s)

Other

Foliation: Foliation planes are parallel planes of different minerals forming a banded appearance on the rock. The foliation planes are usually of a different color than the surrounding rock. Also the rock shears along the foliation planes if struck with a hammer. Record the following:

Close spaced (CS) – 1/8" (3mm) or closer; Medium spaced (MS) – 1/8" to 1/4" (3mm to 6mm); Open spaced (OS) – 1/4" (6mm) or larger

The angle to the horizontal should be measured (with a protractor) and recorded for the rock core. (Several different angles can be found in each 5' to 10' core.)

Weathering: Use the proper number 1 through 5.

1. Unweathered: No evidence of any mechanical or chemical alteration along discoloration evidenced.
2. Slightly weathered: Discoloration is evident, on surface, slight alteration no discontinuities, less than 10% of the volume is altered, strength is substantially unaffected.
3. Moderately weathered: Discoloring is evident, surface is pitted and altered with alteration penetrating will below rock surfaces, weathering "halos" evident, 10% to 50% of the rock is altered, strength is noticeably less than fresh rock.
4. Highly weathered: Entire mass is discolored; alteration pervades nearly all of the rock with some pockets of slightly weathered rock noticeable, some minerals leached away, retains only a fraction of original strength (with wet strength usually lower than dry strength).
5. Decomposed: Rock is reduced to a soil with relict rock structure (saprolite), can be generally molded and crumbled by hand.

Recovery Core Recovery

Rock Quality: Use the proper number 1 through 5

1.	Very Poor	RQD = 0% to 25%
2.	Poor	RQD = 25% to 50%
3.	Fair	RQD = 50% to 75%
4.	Good	RQD = 75% to 90%
5.	Excellent	RQD = 90% to 100%

Examples:

1. Moderately hard, blue-gray to gray, weathered **BIOTITE GNEISS BOULDER**, medium texture

   Recovery = 24%
   RQD = 17%

2. Very hard, gray and white, relatively sound to sound **BIOTITE GNEISS**, medium to fine texture, some quartz veins, foliation angle = 20 degrees

   Recovery = 100%
   RQD = 100%

   -Fresh break @ approximately 47'

BORING LOG

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
EASTERN FEDERAL LANDS HIGHWAY DIVISION

Project Name: PRA-BLRI 2P16 Landslide Repair at MP 364.72 | Boring No.: B-1 | Sheet: 1 of 1

Project Location: Blue Ridge Parkway, Buncombe County, North Carolina | Boring Location: South of the slide

Groundwater Depth:	Surface Elevation: 5492.5 ft	Boring Began: 1/8/08	Completed: 1/9/08
Encountered at:	Caved at:	Boring Method: HSA	Inspector: M.A & A.R
At Completion:	Hammer Wt. & Type: 140 lbs/Automatic	Hole Diameter: 3.8 in.	Operator: B.K & D.H
After ___ hrs	Hammer Drop: 30 in.	Rock Core Diam: 2	Weather: Cold & Cloudy

▼ Water Content %
Plastic Limit ├──┤ Liquid Limit
● Standard Penetration Test Data (Blows / ft): 10 20 40 60 80

Elevation (feet)	Layer Depth (ft)	MATERIAL DESCRIPTION (Density, Color, Plasticity, Size, Proportions, Moisture)	Sample Type	No.	Rec.	Blows per 6 in.	SPT (Blows/ft)
5487.9	4.6	Loose, brown, Micaceous **SILTY SAND**, trace to little clay and rock fragments **(Moist)** **(Fill)** Auger Refusal at 4.6 ft	SPT	J-1		1-4-4-6	●
5483.9	8.6	Highly weathered rock, light gray to gray, **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:Moderate (4) R : 58% RQD : 11%	Rock Core				
5478.9	13.6	Weathered rock, light gray to gray, **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:Moderate (3) R : 52% RQD : 39%	Rock Core				
5475.5	17.0	Weathered rock, light gray to gray, **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:Moderate (3) R : 68% RQD : 49%	Rock Core				
5468.9	23.6	Bed rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Hard (H) Weathering:Slight (2) R : 100% RQD : 59%	Rock Core				
5463.9	28.6	Light gray to gray, **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Hard (H) Weathering:Slight (2) R : 100% RQD : 83%	Rock Core				

Rock coring was terminated at a depth of 28.6 ft

Depth Scale (ft): 5, 10, 15, 20, 25, 30

Sample Types: Auger Cuttings, Vane Shear, SPT, UD, Penetrometer, Rock Core

Remarks:
Competent rock was encountered at a depth of 17 ft below existing site grade.
R: Recovery (%)
RQD: Rock Quality Designation (%)

BORING LOG

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
EASTERN FEDERAL LANDS HIGHWAY DIVISION

Project Name: PRA-BLRI 2P16 Landslide Repair at MP 364.72 | Boring No.: B-2 | Sheet: 1 of 1

Project Location: Blue Ridge Parkway, Buncombe County, North Carolina | Boring Location: Slide area, north of the collapsed pipe culvert

Groundwater Depth:	Surface Elevation: 5493.5 ft	Boring Began:	1/9/08	Completed:	1/9/08
Encountered at:	Caved at:	Boring Method:	HSA	Inspector:	M.A & A.R
At Completion:	Hammer Wt. & Type: 140 lbs/Automatic	Hole Diameter:	3.8 in.	Operator:	B.K & D.H
After 24 hrs 10.0 ft	Hammer Drop: 30 in.	Rock Core Diam:	2	Weather:	Cold & Cloudy

Elevation (feet)	Layer Depth (ft)	MATERIAL DESCRIPTION (Density, Color, Plasticity, Size, Proportions, Moisture)	Sample Type	No.	Rec.	Blows per 6 in.
		Loose to medium dense, brown, Micaceous **SILTY SAND** trace to little clay and rock fragments **(Moist)** **(Fill)**	SPT	J-1		3-6-11-9
			SPT	J-2		23-7-13-15
		(water level at 10 ft)	SPT	J-3		0-2-4-8
5479.8	13.7	Auger Refusal at 13.7 ft				
5478.5	15.0	Weathered rock, light gray to gray **MICACEOUS SCHIST**	Rock Core			
5474.8	18.7	Bed rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Hard (H) Weathering:Slight (2) R : 100% RQD : 75%	Rock Core			
5469.8	23.7	Light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Hard (H) Weathering:Slight (2) R : 97% RQD : 50%	Rock Core			
		Rock coring was terminated at a depth of 23.7 ft				

Depth Scale (ft): 5, 10, 15, 20, 25, 30

▼ Water Content %
Plastic Limit ├──┤ Liquid Limit
● Standard Penetration Test Data (Blows / ft)
10 20 40 60 80

Sample Types:
- Auger Cuttings
- Vane Shear
- SPT
- UD
- Penetrometer
- Rock Core

Remarks:
Competent rock was encountered at a depth of 15 ft below existing site grade.
R: Recovery (%)
RQD: Rock Quality Designation (%)

BORING LOG

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
EASTERN FEDERAL LANDS HIGHWAY DIVISION

Project Name: PRA-BLRI 2P16 Landslide Repair at MP 364.72 | Boring No.: B-3 | Sheet: 1 of 2

Project Location: Blue Ridge Parkway, Buncombe County, North Carolina | Boring Location: North of the slide

Groundwater Depth:	Surface Elevation: 5494.0 ft	Boring Began:	1/10/08	Completed:	1/11/08
Encountered at:	Caved at:	Boring Method:	HSA	Inspector:	M.A & A.R
At Completion:	Hammer Wt. & Type: 140 lbs/Automatic	Hole Diameter:	3.8 in.	Operator:	B.K & D.H
After ___ hrs	Hammer Drop: 30 in.	Rock Core Diam:	2	Weather:	Cold & Cloudy

Water Content % ▼; Plastic Limit ├──┤ Liquid Limit; ● Standard Penetration Test Data (Blows / ft): 10, 20, 40, 60, 80

Elevation (feet)	Layer Depth (ft)	MATERIAL DESCRIPTION (Density, Color, Plasticity, Size, Proportions, Moisture)	Sample Type	No.	Rec.	Blows per 6 in.
5490.5	3.5	Very loose, brown, micaceous **SILTY SAND** trace to little clay and rock fragments **(Moist)** **(Fill)** Auger Refusal at 3.5 ft.	SPT	J-1		1-1-1-2
5485.5	8.5	Weathered rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:Moderate (3) R : 30% RQD : 20%	Rock Core			
5480.5	13.5	Highly weathered rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:High (4) R : 40% RQD : 23%	Rock Core			
5475.5	18.5	Highly weathered rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:High (4) R : 40% RQD : 23%	Rock Core			
5470.5	23.5	Weathered rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:Moderate (3) R : 40% RQD : 26%	Rock Core			
5467.0	27.0	Weathered rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:Moderate (3) R : 70% RQD : 54%	Rock Core			
5464.0	30.0	Bed rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Hard (H)	Rock Core			

Depth Scale (ft): 5, 10, 15, 20, 25, 30

Sample Types: Auger Cuttings, Vane Shear, SPT, UD, Penetrometer, Rock Core

Remarks:
Competent rock was encountered at a depth of 27 ft below existing site grade.
R: Recovery (%)
RQD: Rock Quality Designation (%)

BORING LOG

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
EASTERN FEDERAL LANDS HIGHWAY DIVISION

Project Name: PRA-BLRI 2P16 Landslide Repair at MP 364.72 Boring No.: B-3 Sheet: 2 of 2

Project Location: Blue Ridge Parkway, Buncombe County, North Carolina Boring Location: North of the slide

Groundwater Depth:	Surface Elevation: 5494.0 ft	Boring Began:	1/10/08	Completed:	1/11/08
Encountered at:	Caved at:	Boring Method:	HSA	Inspector:	M.A & A.R
At Completion:	Hammer Wt. & Type: 140 lbs/Automatic	Hole Diameter:	3.8 in.	Operator:	B.K & D.H
After ___ hrs	Hammer Drop: 30 in.	Rock Core Diam:	2	Weather:	Cold & Cloudy

Elevation (feet)	Graphic Log	Layer Depth (ft)	MATERIAL DESCRIPTION Density, Color, Plasticity, Size, Proportions, Moisture	Depth Scale (ft)	SAMPLE Type	No.	Rec.	Blows per 6 in.	▼ Water Content % / Plastic Limit ⊢—⊣ Liquid Limit / ● Standard Penetration Test Data (Blows / ft) 10 20 40 60 80
			Weathering:Slight (2) R : 100% RQD : 95%		Rock Core				
5460.5		33.5	Light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Hard (H) Weathering:Slight (2) R : 100% RQD : 95%						
			Rock coring was terminated at a depth of 33.5 ft	35					
				40					
				45					
				50					
				55					
				60					

Sample Types:
- Auger Cuttings
- Vane Shear
- SPT
- UD
- Penetrometer
- Rock Core

Remarks:
Competent rock was encountered at a depth of 27 ft below existing site grade.
R: Recovery (%)
RQD: Rock Quality Designation (%)

BORING LOG

U. S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION
EASTERN FEDERAL LANDS HIGHWAY DIVISION

Project Name: PRA-BLRI 2P16 Landslide Repair at MP 364.72 — Boring No.: B-4 — Sheet: 1 of 1

Project Location: Blue Ridge Parkway, Buncombe County, North Carolina — Boring Location: Opposite side of the road, slide area

Groundwater Depth:	Surface Elevation: 5496.0 ft	Boring Began: 1/10/08	Completed: 1/11/08
Encountered at:	Caved at:	Boring Method: HSA	Inspector: M.A & A.R
At Completion:	Hammer Wt. & Type: 140 lbs/Automatic	Hole Diameter: 3.8 in.	Operator: B.K & D.H
After hrs	Hammer Drop: 30 in.	Rock Core Diam: 2	Weather: Cold & Cloudy

Legend: ▼ Water Content %; Plastic Limit ├──┤ Liquid Limit; ● Standard Penetration Test Data (Blows / ft) — scale 10, 20, 40, 60, 80

Elevation (feet)	Graphic Log	Layer Depth (ft)	MATERIAL DESCRIPTION (Density, Color, Plasticity, Size, Proportions, Moisture)	Depth Scale (ft)	Sample Type	No.	Rec.	Blows per 6 in.
5493.0		3.0	Medium dense, gray, micaceous **SILTY SAND**, trace to little clay and rock fragments **(Moist)** **(Fill)** Auger refusal at 3 ft		SPT	J-1		1-5-15-30
5487.0		9.0	Highly weathered rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:Moderate (4) R : 100% RQD : 0%	5	Rock Core			
5483.0		13.0	Weathered rock, light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Medium Hard (MH) Weathering:Moderate (3) R : 100% RQD : 64%	10	Rock Core			
5477.0		19.0	Bed rock , light gray to gray **MICACEOUS SCHIST** Texture: Fine (F) Hardness: Hard (H) Weathering:Slight (2) R : 100% RQD : 93%	15	Rock Core			
			Rock coring was terminated at 19 ft	20, 25, 30				

Sample Types: Auger Cuttings; Vane Shear; SPT; UD; Penetrometer; Rock Core

Remarks:
Competent rock was encountered at a depth of 13 ft below existing site grade.
R: Recovery (%)
RQD: Rock Quality Designation (%)

APPENDIX D
Geophysical Report

BLACKHAWK
A DIVISION OF
ZAPATAENGINEERING

February 14, 2008

Mounir Abouzakhm
Federal Highway Administration
Eastern Federal Land Highway Division
21400 Ridgetop Circle
Sterling, Virginia 20166

Re: Summary of Seismic Refraction Data Processing for Blue Ridge Parkway, North Carolina.

The following letter provides a brief report on the data processing for the seismic refraction data sets collected by Eastern Federal Lands Highway Division (EFLHD) Blue Ridge Parkway, NC. The data includes two lines with a total of two seismic refraction spreads. Data were collected using a 24-channel receiver array on all lines. The array geometry is outlined in the observer's notes provided by EFLHD. Additional information including seismograph, geophones, etc. was not provided, but is typically not critical for data processing.

ZAPATAENGINEERING, Blackhawk Division (Blackhawk) performed the data reduction and processing only. Blackhawk did not collect the data or oversee data collection. All field notes, seismic data, and land survey data were provided by EFLHD.

Data Processing:

The seismic data were processed using the Generalized Reciprocal Method (GRM) and the refraction tomography method.

The general data processing flow for the GRM is outlined below:

- Import each shot record into Pickwin95 (Oyo SeisImager software)
- Apply geometry corrections and save file (SeisImager internal format)
- Pick first arrival data for each shot record
- Apply 250 Hz high cut filter to mute high frequency ambient noise, if necessary
- Repick first arrival data as necessary noting any phase shift
- Export first break pick file for each spread
- Create ASCII elevation file in appropriate format
- Import first break pick file and elevation file into Plotrefa (Oyo SeisImager software)
- Static shift time-distance curves to correct for reciprocal time error

- Import first break pick file into Visual Basic
- Convert into Interpex Ltd, Gremix format
- Import first break pick file into Gremix
- Input station elevations
- Assign first arrivals to layers
- Check reciprocal times
- Analyze data using Optimum GRM processing
- Review and edit timedepth data
- Produce DXF of results plot
- Import DXF into AutoCAD

The general data processing flow for the refraction tomography is outlined below:

- Import each shot record into Pickwin95 (Oyo SeisImager software)
- Truncate record length.
- Apply 2D geometry corrections and save file (SeisImager internal format)
- Pick first arrival data for each shot record
- Create ASCII elevation file in appropriate format
- Import first break pick file and elevation file into Plotrefa (Oyo SeisImager software)
- Create initial tomography model
- Run refraction tomography inversion
- Trim and rescale image based on raypath data
- Screen capture and export velocity tomogram
- Import tomograms into CorelDraw to create figures

General Data Considerations:

It should be noted that the quality of the data is fair which made it difficult to pick the first arrivals. In addition, it appears that on some of the records, a high velocity surface layer exists, further complicating the interpretation of the data. This layer may represent the hard surface layer along a road. Since the data was recorded in a valley it is not certain that the first refractor is underneath the seismic spread or to one side of the spread and it may come from a dipping layer. In this case the depths are those orthogonal to the refractor dip and are not vertical depths to the refractor beneath the spread.

Individual Seismic Refraction Lines:

Each seismic refraction line is shown as a separate figure. Line 0001 is shown on Figure 1, while Line 001 is shown on Figure 2. Each figure contains three GRM plot windows with the tomography results displayed on the middle GRM window. The upper window shows the first break pick data with layer fits. The middle window shows the cross sectional profile of the GRM solution. The lower window shows the GRM solution layer P-wave velocities.

Limitations and Recommendations:

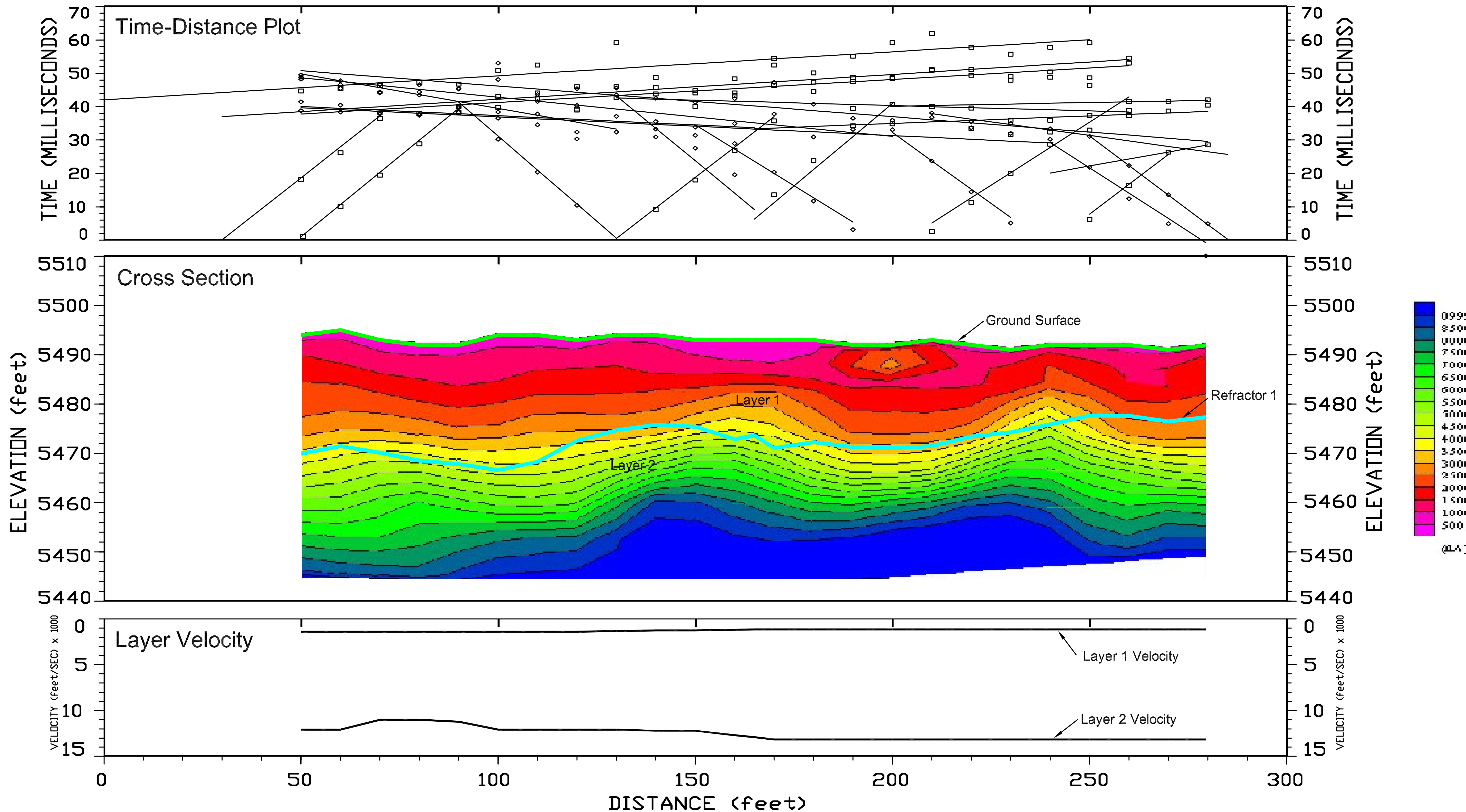
The site was located along a road cut; therefore, the calculated depth to bedrock may represent the closest bedrock to the geophone and may not be in the vertical plane. The depth to the overburden/top of bedrock interface calculated from the seismic data may vary somewhat from boring logs because of the irregular and gradational nature of this interface. The calculated depth of a refractor is primarily controlled by the velocity of the overburden. Lateral variability in V1 required a limiting of the velocity range in the interpretation. Because of the large lateral variation in the overburden velocity, there is potential for significant error in the depth to top of refractor calculation if this velocity variation is not adequately sampled.

Blackhawk appreciates this opportunity to be of service to EFLHD. We appreciate all efforts extended to us in this matter. Please feel free to contact us with any questions you may have.

Sincerely,

Jim Pfeiffer
Associate Geophysicist

Jim Hild
Manager/Sr. Geophysicist



BLACKHAWK
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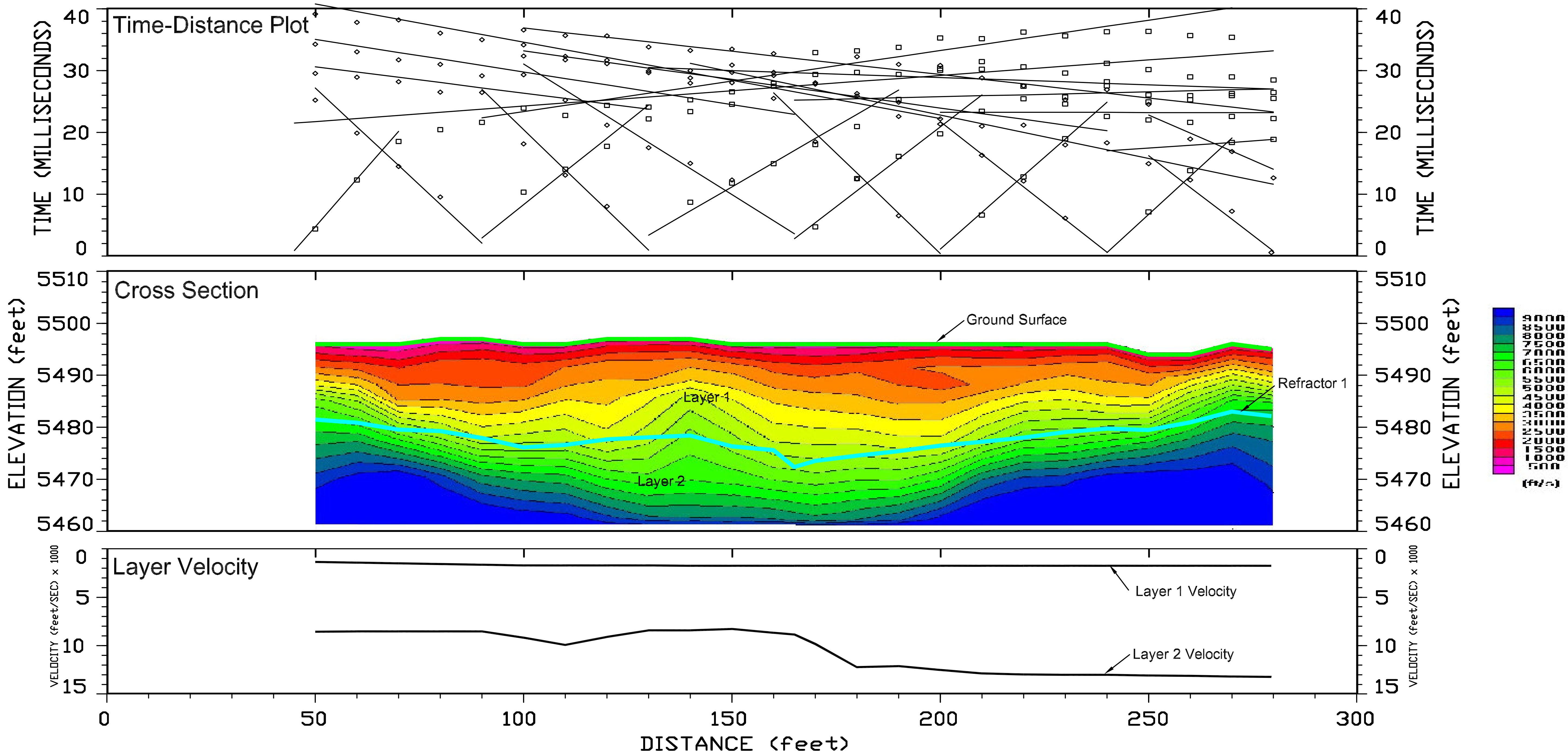
FHWA-EFLHD
Eastern Federal Lands Highway Division
Sterling, Virginia

Seismic Refraction Survey
Line 0001
Blue Ridge Parkway
North Carolina

Project No:	Date:	Drawn By:	Checked By:	Scale:	Figure:
5108	Feb., 2008	HJV	JP	H25' - V20'	1



BLACKHAWK A DIVISION OF ZAPATA ENGINEERING	FHWA-EFLHD *Eastern Federal Lands Highway Division* *Sterling, Virginia*		Seismic Refraction Survey Line 001 *Blue Ridge Parkway* *North Carolina*		
301 Commercial Road, Suite B, Golden, Colorado 80401 — Phone: (303) 278-8700 Fax: (303) 278-0789 Web: www.blackhawkgeo.com	Project No: 5108	Date: Feb., 2008	Drawn By: HJV	Checked By: JP	Scale: H25' - V20'	Figure: 2

APPENDIX E
Laboratory Test Results

Project Name:	Slide at MP 364.4	**State:**	NC
Project Number:	BLRI 2P16		
Field Sample No:	B-1/ 5		
Submitted By:	M. Abouzakhm		

Unconfined Compressive Strength of Intact Rock Core (ASTM 2938)

Unconfined Compressive Strength, psi	4690

Charles W. McCown, Jr. - Laboratory Team Leader

Date

Project Name:	Slide at MP 364.4	**State:**	NC
Project Number:	BLRI 2P16		
Field Sample No:	B-2/ 2		
Submitted By:	M. Abouzakhm		

Unconfined Compressive Strength of Intact Rock Core (ASTM 2938)

Unconfined Compressive Strength, psi	5920

Charles W. McCown, Jr. - Laboratory Team Leader

Date

U.S. Department of Transportation
Federal Highway Administration
Federal Lands Highway Office

Eastern Federal Lands Highway Division
Materials Testing Laboratory
112 Industrial Park, Sevierville, TN 37862

Project Name:	Slide at MP 364.4	**State:**	NC
Project Number:	BLRI 2P16		
Field Sample No:	B-3/ 4		
Submitted By:	M. Abouzakhm		

Unconfined Compressive Strength of Intact Rock Core (ASTM 2938)

Unconfined Compressive Strength, psi	3040

Charles W. McCown, Jr. - Laboratory Team Leader

Date

APPENDIX F
Slope Stability Design Analysis (ReSSA)

Slide at MP 364.72

Report created by ReSSA(2.0): Copyright (c) 2001-2005, ADAMA Engineering, Inc.

PROJECT IDENTIFICATION

Title:	Slide at MP 364.72
Project Number:	BLRI 2P16 -
Client:	Blue Ridge Parkway
Designer:	M.A.A
Station Number:	Sta. 11+25

Description:
Retaining Wall Slide

Company's information:

Name:	EFLHD
Street:	21400 Ridgetop Circle Sterling, VA 20166
Telephone #:	(571) 434-1566
Fax #:	(703) 404-6217
E-Mail:	Mounir.Abouzakhm@fhwa.dot.gov

Original file path and name: M:\Project ..... \Analysis (2P16)\Reinf Embankment no key _rock.MSE
Original date and time of creating this file: Thu Apr 10 12:52:53 2008

PROGRAM MODE: ANALYSIS of a Complex Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1.......Embankment...	130.0	32.0	0.0
....2.......Ret Wall..	130.0	34.0	0.0
....3.......Retained Soil..	145.0	40.0	1200.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Coverage Ratio, Rc
1	Geosynthetic	9000.00	1.20	1.10	1.67	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
1	Geosynthetic	0.80	0.00	0.80	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50

WATER

Water is not present

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.

GEOMETRY

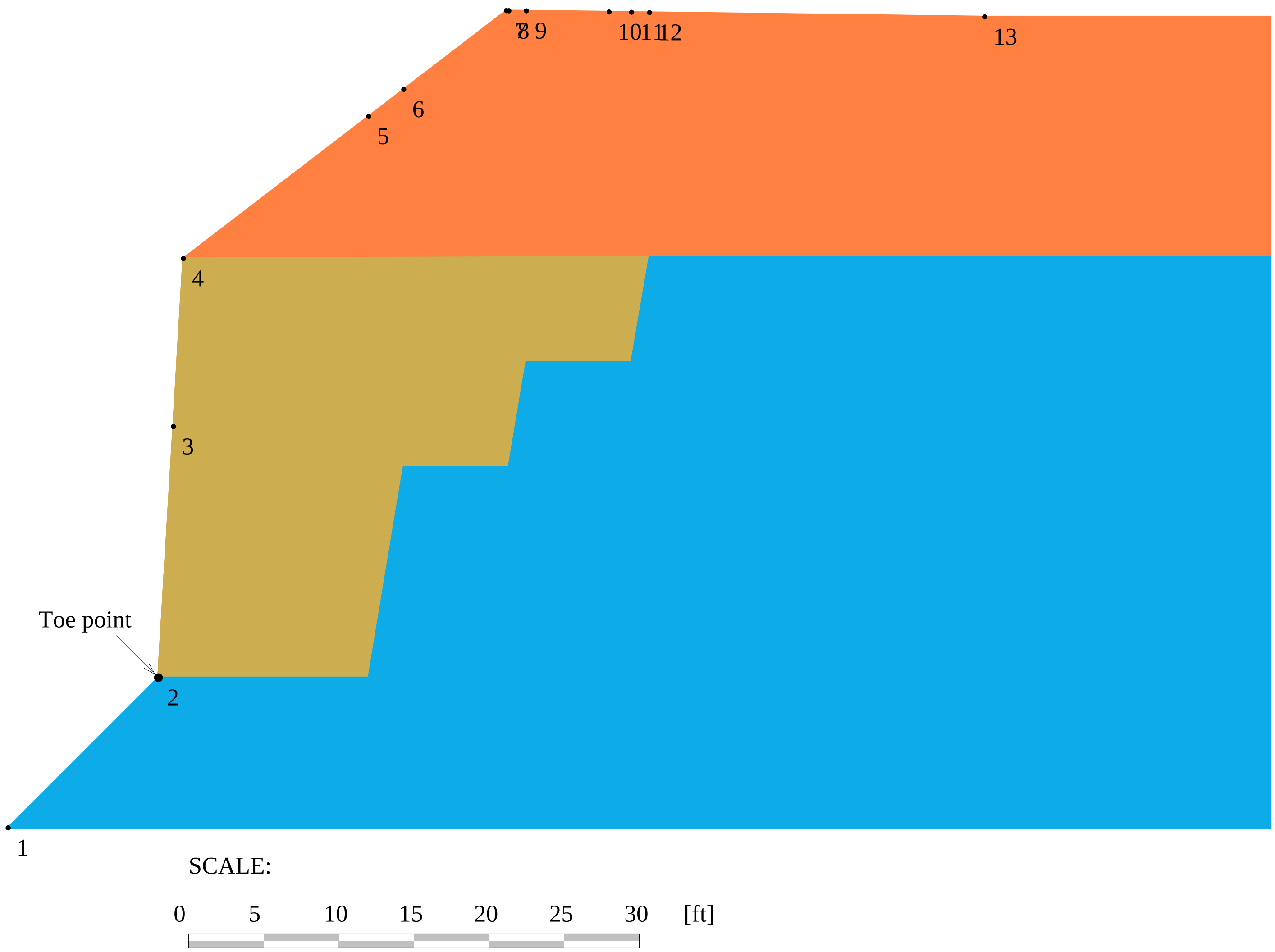
Soil profile contains 3 layers (see details in next page)

UNIFORM SURCHARGE

Surcharge load, Q1.................................... None
Surcharge load, Q2.................................... None
Surcharge load, Q3.................................... None

STRIP LOAD

............................None..................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

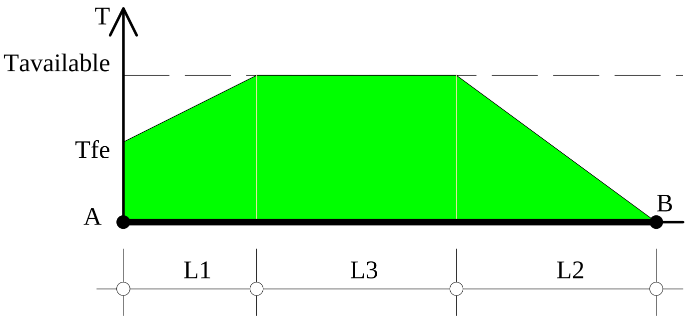
	#	Xi	Yi
Top of Layer 1	1	40.00	40.00
	2	50.00	50.00
	3	51.67	77.91
	4	73.17	94.41
	5	105.00	94.00
Top of Layer 2	6	40.00	40.00
	7	50.00	50.00
	8	51.67	77.91
	9	80.00	78.00
Top of Layer 3	10	40.00	40.00
	11	50.00	50.00
	12	51.00	50.00
	13	64.00	50.00
	14	66.33	64.00
	15	73.33	64.00
	16	74.50	71.00
	17	81.50	71.00
	18	82.70	78.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 3 layers. Coordinates in [ft.]

#	X	Y1	Y2	Y3
1	40.00	40.00	40.00	40.00
2	50.00	50.00	50.00	50.00
3	51.00	66.71	66.71	50.00
4	51.67	77.91	77.91	50.00
5	64.00	87.37	77.95	50.00
6	66.33	89.16	77.96	64.00
7	73.17	94.41	77.98	64.00
8	73.33	94.41	77.98	64.00
9	74.50	94.39	77.98	71.00
10	80.00	94.32	78.00	71.00
11	81.50	94.30	78.00	71.00
12	82.70	94.29	78.00	78.00
13	105.00	94.00	78.00	78.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	Geosynthetic	0.60	14.00	0.00	1.51	12.49	4082.74	4082.74
2	Geosynthetic	2.10	14.00	0.00	1.57	12.43	4082.74	4082.74
3	Geosynthetic	3.60	14.00	0.00	1.64	12.36	4082.74	4082.74
4	Geosynthetic	5.10	14.00	0.00	1.74	12.26	4082.74	4082.74
5	Geosynthetic	6.60	14.00	0.00	1.80	12.20	4082.74	4082.74
6	Geosynthetic	8.10	14.00	0.00	1.90	12.10	4082.74	4082.74
7	Geosynthetic	9.60	14.00	0.00	2.00	12.00	4082.74	4082.74
8	Geosynthetic	11.10	14.00	0.00	2.13	11.87	4082.74	4082.74
9	Geosynthetic	12.60	14.00	0.00	2.23	11.77	4082.74	4082.74
10	Geosynthetic	14.10	21.00	0.00	1.94	19.06	4082.74	4082.74
11	Geosynthetic	15.60	21.00	0.00	2.03	18.97	4082.74	4082.74
12	Geosynthetic	17.10	21.00	0.00	2.13	18.87	4082.74	4082.74
13	Geosynthetic	18.60	21.00	0.00	2.26	18.74	4082.74	4082.74
14	Geosynthetic	20.10	21.00	0.00	2.43	18.57	4082.74	4082.74
15	Geosynthetic	21.60	27.00	0.00	2.43	24.57	4082.74	4082.74
16	Geosynthetic	23.10	27.00	0.00	2.59	24.41	4082.74	4082.74
17	Geosynthetic	24.60	27.00	0.00	2.79	24.21	4082.74	4082.74
18	Geosynthetic	26.10	27.00	0.00	3.02	23.98	4082.74	4082.74
19	Geosynthetic	27.60	27.00	0.00	3.28	23.72	4082.74	4082.74
20	Geosynthetic	29.10	15.00	0.00	6.56	8.44	4082.74	4082.74
21	Geosynthetic	30.60	15.00	0.00	6.56	8.44	4082.74	4082.74
22	Geosynthetic	32.10	15.00	0.00	6.56	8.44	4082.74	4082.74
23	Geosynthetic	33.60	15.00	0.00	6.63	8.37	4082.74	4082.74
24	Geosynthetic	35.10	15.00	0.00	7.09	7.91	4082.74	4082.74
25	Geosynthetic	36.60	15.00	0.00	8.10	6.90	4082.74	4082.74
26	Geosynthetic	38.10	15.00	0.00	10.04	4.96	4082.74	4082.74
27	Geosynthetic	39.60	15.00	0.00	15.00	0.00	3917.11	3917.11 (*)
28	Geosynthetic	41.10	15.00	0.00	15.00	0.00	2892.30	2892.30 (*)
29	Geosynthetic	42.60	15.00	0.00	15.00	0.00	1660.85	1660.85 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified ultimate strength

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	72.97	94.26	41.69	41.83	12.86	94.58	60.11	5.86	
2	75.59	94.38	41.99	42.00	17.41	94.73	58.18	3.39	
3	78.22	94.35	41.65	41.87	21.77	94.70	56.45	2.34	
4	80.84	94.31	41.89	41.97	25.95	94.49	54.89	1.84	
5	83.47	94.28	41.92	41.98	23.36	99.37	60.32	1.73	
6	86.09	94.24	41.89	41.96	16.60	108.16	70.87	1.66	
7	88.72	94.21	41.85	41.93	7.21	120.13	85.52	1.62	
. 8	91.34	94.18	41.86	41.93	-2.74	133.72	102.05	1.60 .	OK
9	93.96	94.14	41.83	41.91	-22.36	158.10	132.74	1.64	
10	96.59	94.11	41.89	41.93	-37.84	180.29	159.68	1.68	
11	99.21	94.07	42.29	42.46	-57.91	210.14	195.34	1.73	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

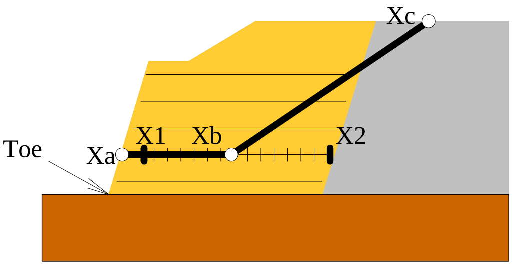
Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
. 1	41.86	41.93	91.34	94.18	-2.74	133.72	102.05	1.60 .	On extreme X-exit
2	42.22	42.47	91.34	94.18	-1.37	133.06	100.54	1.61	
3	43.24	43.32	91.34	94.18	-0.01	132.41	99.03	1.61	
4	43.63	43.87	91.34	94.18	1.35	131.75	97.52	1.61	
5	44.65	44.73	91.34	94.18	2.71	131.09	96.01	1.62	
6	45.06	45.29	91.34	94.18	4.07	130.43	94.50	1.62	
7	46.11	46.16	91.34	94.18	8.63	126.78	88.91	1.63	
8	46.54	46.73	91.34	94.18	9.93	126.17	87.47	1.63	
9	47.58	47.59	91.34	94.18	11.24	125.57	86.03	1.64	
10	48.03	48.17	91.34	94.18	12.55	124.97	84.59	1.64	
11	48.50	48.77	91.34	94.18	13.86	124.37	83.16	1.65	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF TRANSLATIONAL ANALYSIS



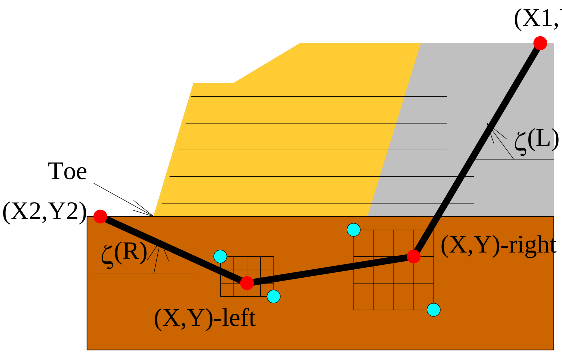
Results in the table below represent critical two-part wedges identified between specified starting (X1) and ending (X2) search points. Wedges along all reinforcement layers and at elevation zero are reported. The critical two-part wedge, one for each predetermined elevation, is defined by Xa, Xb and Xc where Xa is the front end of the passive wedge (slope face), Xb is where the passive wedge ends and the active one starts, and Xc is the X-ordinate at which the active wedge starts.

Critical two-part wedge along each interface:

Interface	Height Relative to Toe [ft]	(Xa, Ya) [ft]		(Xb, Yb) [ft]		(Xc, Yc) [ft]		Fs	STATUS
At toe elevation	0.00	50.00	50.00	50.20	50.00	108.59	94.00	1.45	Minimum on Edge
. Reinf. Layer #1	0.60	50.04	50.60	53.18	50.60	101.43	94.05	1.41	OK
Reinf. Layer #2	2.10	50.13	52.10	56.06	52.10	101.04	94.05	1.47	OK
Reinf. Layer #3	3.60	50.22	53.60	58.83	53.60	102.20	94.04	1.54	OK
Reinf. Layer #4	5.10	50.31	55.10	56.16	55.10	104.21	94.01	1.55	OK
Reinf. Layer #5	6.60	50.39	56.60	56.26	56.60	109.67	94.00	1.65	OK
Reinf. Layer #6	8.10	50.48	58.10	61.91	58.10	104.70	94.00	1.65	OK
Reinf. Layer #7	9.60	50.57	59.60	62.01	59.60	104.50	94.01	1.62	OK
Reinf. Layer #8	11.10	50.66	61.10	64.89	61.10	100.24	94.06	1.71	Minimum on Edge
Reinf. Layer #9	12.60	50.75	62.60	64.99	62.60	106.66	94.00	1.76	Minimum on Edge
Reinf. Layer #10	14.10	50.84	64.10	63.73	64.10	109.78	94.00	1.93	OK
Reinf. Layer #11	15.60	50.93	65.60	68.01	65.60	103.11	94.02	1.99	OK
Reinf. Layer #12	17.10	51.02	67.10	68.11	67.10	107.99	94.00	2.07	OK
Reinf. Layer #13	18.60	51.11	68.60	72.29	68.60	102.60	94.03	2.09	Minimum on Edge
Reinf. Layer #14	20.10	51.20	70.10	55.68	70.10	90.09	94.19	2.18	OK
Reinf. Layer #15	21.60	51.29	71.60	56.98	71.60	90.46	94.19	2.18	OK
Reinf. Layer #16	23.10	51.38	73.10	62.46	73.10	89.46	94.20	2.14	OK
Reinf. Layer #17	24.60	51.47	74.60	62.56	74.60	88.58	94.21	2.06	OK
Reinf. Layer #18	26.10	51.56	76.10	62.66	76.10	91.58	94.17	2.01	OK
Reinf. Layer #19	27.60	51.65	77.60	68.13	77.60	87.94	94.22	1.96	OK
Reinf. Layer #20	29.10	53.22	79.10	68.39	79.10	87.75	94.22	1.61	Minimum on Edge
Reinf. Layer #21	30.60	55.18	80.60	70.39	80.60	87.82	94.22	1.77	Minimum on Edge
Reinf. Layer #22	32.10	57.13	82.10	72.39	82.10	87.90	94.22	1.98	Minimum on Edge
Reinf. Layer #23	33.60	59.08	83.60	74.29	83.60	87.88	94.22	2.27	Minimum on Edge
Reinf. Layer #24	35.10	61.04	85.10	76.29	85.10	87.96	94.22	2.66	Minimum on Edge
Reinf. Layer #25	36.60	62.99	86.60	78.19	86.60	87.94	94.22	3.14	Minimum on Edge
Reinf. Layer #26	38.10	64.95	88.10	77.21	88.10	87.41	94.23	4.10	OK
Reinf. Layer #27	39.60	66.90	89.60	76.13	89.60	84.22	94.27	5.34	OK
Reinf. Layer #28	41.10	68.86	91.10	75.16	91.10	80.72	94.31	7.06	OK
Reinf. Layer #29	42.60	70.81	92.60	74.08	92.60	75.19	94.38	8.73	OK

Note: In the 'Status' column, OK means the critical two part-wedge was identified within the specified search domain. 'Minimum on Edge' means the critical result corresponds to a minimum on the edge of the search domain; i.e., either on X1 or X2 or the internally preset limits on Xc.

RESULTS OF 3-PART WEDGE ANALYSIS



Results in the table below represent the critical slip surface composed of a three-part wedge and identified by the specified points (X-left, Y-left) and (X-right, Y-right) and angles Zeta(L) and Zeta(R). ReSSA finds the (X,Y) coordinates, as well as the angles Zeta, based on user-specified search domain. The trace of the critical three-part wedge is fully defined by four points: (X1, Y1), (X-left, Y-left), (X-right, Y-right), (X2, Y2).

Critical 3-part wedge (Automatic search):

(X2, Y2) [ft]	Zeta(L) [degrees]	(X-left, Y-left) [ft]	(X-right, Y-right) [ft]	Zeta(R) [degrees]	(X1, Y1) [ft]	Fs
(50.00, 50.00)	0.00	(50.00, 50.00)	(50.00, 50.00)	0.00	(50.00, 50.00)	1000044.400

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.60

Critical Circle: Xc = -2.74[ft], Yc = 133.72[ft], R = 102.05[ft]. (Number of slices used = 61)

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.41

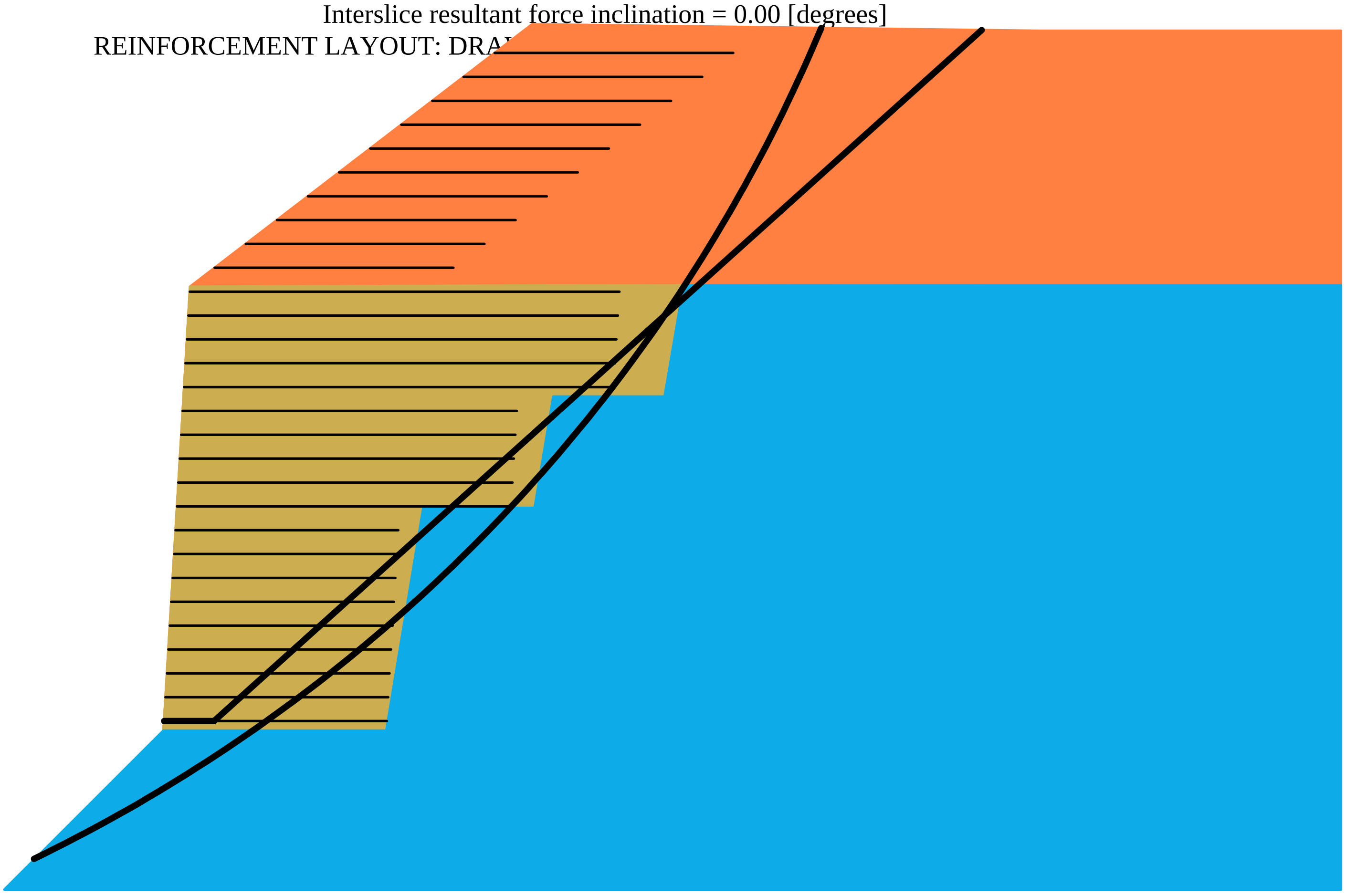
Critical Two-Part Wedge: (Xa = 50.04, Ya = 50.60) [ft]
(Xb = 53.18, Yb = 50.60) [ft]
(Xc = 101.43, Yc = 94.05) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 41.43 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1000044.40

Critical Three-Part Wedge: (X2 = 50.00, Y2 = 50.00) [ft]
(X-left = 50.00, Y-left = 50.00) [ft]
(X-right = 50.00, Y-right = 50.00) [ft]
(X1 = 50.00, Y1 = 50.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 0.00 [degrees]

REINFORCEMENT LAYOUT: DRA



SCALE:

0 5 10 15 20 25 30 [ft]

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES

Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]	
1	1	Geosynthetic	0.60	14.00	1.00	164.08	164.64	178.08	164.64
2	1	Geosynthetic	2.10	14.00	1.00	164.17	166.14	178.17	166.14
3	1	Geosynthetic	3.60	14.00	1.00	164.26	167.64	178.26	167.64
4	1	Geosynthetic	5.10	14.00	1.00	164.35	169.14	178.35	169.14
5	1	Geosynthetic	6.60	14.00	1.00	164.44	170.64	178.44	170.64
6	1	Geosynthetic	8.10	14.00	1.00	164.53	172.14	178.53	172.14
7	1	Geosynthetic	9.60	14.00	1.00	164.62	173.64	178.62	173.64
8	1	Geosynthetic	11.10	14.00	1.00	164.71	175.14	178.71	175.14
9	1	Geosynthetic	12.60	14.00	1.00	164.80	176.64	178.80	176.64
10	1	Geosynthetic	14.10	21.00	1.00	164.89	178.14	185.89	178.14
11	1	Geosynthetic	15.60	21.00	1.00	164.98	179.64	185.98	179.64
12	1	Geosynthetic	17.10	21.00	1.00	165.07	181.14	186.07	181.14
13	1	Geosynthetic	18.60	21.00	1.00	165.15	182.64	186.15	182.64
14	1	Geosynthetic	20.10	21.00	1.00	165.24	184.14	186.24	184.14
15	1	Geosynthetic	21.60	27.00	1.00	165.33	185.64	192.33	185.64
16	1	Geosynthetic	23.10	27.00	1.00	165.42	187.14	192.42	187.14
17	1	Geosynthetic	24.60	27.00	1.00	165.51	188.64	192.51	188.64
18	1	Geosynthetic	26.10	27.00	1.00	165.60	190.14	192.60	190.14
19	1	Geosynthetic	27.60	27.00	1.00	165.69	191.64	192.69	191.64
20	1	Geosynthetic	29.10	15.00	1.00	167.26	193.14	182.26	193.14
21	1	Geosynthetic	30.60	15.00	1.00	169.22	194.64	184.22	194.64
22	1	Geosynthetic	32.10	15.00	1.00	171.17	196.14	186.17	196.14
23	1	Geosynthetic	33.60	15.00	1.00	173.13	197.64	188.13	197.64
24	1	Geosynthetic	35.10	15.00	1.00	175.08	199.14	190.08	199.14
25	1	Geosynthetic	36.60	15.00	1.00	177.04	200.64	192.04	200.64
26	1	Geosynthetic	38.10	15.00	1.00	178.99	202.14	193.99	202.14
27	1	Geosynthetic	39.60	15.00	1.00	180.94	203.64	195.94	203.64
28	1	Geosynthetic	41.10	15.00	1.00	182.90	205.14	197.90	205.14
29	1	Geosynthetic	42.60	15.00	1.00	184.85	206.64	199.85	206.64

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	Geosynthetic	1.00	516.00

APPENDIX G
Anchor Design Calculations

BLRI 2P16
Wall Anchor Design
Wall slide Stabilization, MP 364.72

GEC 4

5.3 - Anchor bond zone must be located sufficiently behind the critical potential failure surface so that load is not transferred from the anchor bond zone into the "no-load" zone.

No Load - unbonded length: Typically a distance of H/5 or 1.5 meters (5 ft) behind critical potential failure surface.

For walls constructed in cohesionless soils, the critical potential failure surface can be assumed to extend up from the corner of the excavation at an angle of $45^{o} + \Phi/2$.

Anchor Design:

5.3.4 Design of Unbonded length

$min_{strand} := 15ft$

$min_{bar} := 10ft$

Minimum lengths of unbonded lengths for strand and bar tendons

$L_{unbond\_crit} := 5ft$

Minimum length of unbonded zone beyond critical failure zone (or H/5)

5.3.6 Design of the anchor bond length

Typical Anchor characteristics:

1. Design Load between 260 and 1160 kN (58 to 260 kips)
2. Total anchor length between 30 and 60 ft.
3. Anchors installed between 10 and 45 degrees from horizontal, 15 to 30 degrees common

First step: Assume maximum anchor bond length of 25 ft. for rock and 15 degree inclination

$kips := 1000lbf$

Calculation of Bond Length:

Rock-grout length:

$d_0 := 3in$ $d_1 := 4in$ $d_2 := 5in$ $d_3 := 6in$ Possible drill hole diameters

Load per anchor

$Q := 35kips$

From GEC-4 (Table 7 assumes competent rock):

Table 7. Presumptive average ultimate bond stress for ground/grout interface along anchor bond zone (after PTI, 1996).

Rock		Cohesive Soil		Cohesionless Soil	
Rock type	Average ultimate bond stress (MPa)	Anchor type	Average ultimate bond stress (MPa)	Anchor type	Average ultimate bond stress (MPa)
Granite and basalt	1.7 - 3.1	Gravity-grouted anchors (straight shaft)	0.03 - 0.07	Gravity-grouted anchors (straight shaft)	0.07 - 0.14
Dolomitic limestone	1.4 - 2.1	Pressure-grouted anchors (straight shaft)		Pressure-grouted anchors (straight shaft)	
Soft limestone	1.0 - 1.4	• Soft silty clay	0.03 - 0.07	• Fine-med. sand, med. dense – dense	0.08 - 0.38
Slates and hard shales	0.8 - 1.4	• Silty clay	0.03 - 0.07	• Med.–coarse sand (w/gravel), med. dense	0.11 - 0.66
Soft shales	0.2 - 0.8	• Stiff clay, med. to high plasticity	0.03 - 0.10	• Med.–coarse sand (w/gravel), dense - very dense	0.25 - 0.97
Sandstones	0.8 - 1.7	• Very stiff clay, med. to high plasticity	0.07 - 0.17	• Silty sands	0.17 - 0.41
Weathered Sandstones	0.7 - 0.8	• Stiff clay, med. plasticity	0.10 - 0.25	• Dense glacial till	0.30 - 0.52
Chalk	0.2 - 1.1	• Very stiff clay, med. plasticity	0.14 - 0.35	• Sandy gravel, med. dense-dense	0.21 - 1.38
Weathered Marl	0.15 - 0.25	• Very stiff sandy silt, med. plasticity	0.28 - 0.38	• Sandy gravel, dense-very dense	0.28 - 1.38
Concrete	1.4 - 2.8				

Note: Actual values for pressure-grouted anchors depend on the ability to develop pressures in each soil type.

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Table 8. Presumptive ultimate values of load transfer for preliminary design of ground anchors in rock.

Rock type	Estimated ultimate transfer load (kN/m)
Granite or Basalt	730
Dolomitic Limestone	580
Soft Limestone	440
Sandstone	440
Slates and Hard Shales	360
Soft Shales	150

Typical ranges of ultimate bond stress values for the rock/grout interface which have been measured are provided in table 7. Alternatively, PTI (1996) suggests that the ultimate bond stress between rock and grout can be approximated as 10 percent of the unconfined compressive strength of the rock up to a maximum value for ultimate bond stress of 3.1 MPa.

GEC refers to PTI (1996) recommendation that ultimate bond stress between rock and grout can be approximated as 10 percent of the unconfined compressive strength of the rock up to a maximum value of 3.1 MPa (450 psi).

LAB TESTING

$$UCRock := \begin{pmatrix} 3040 \\ 4690 \\ 5920 \end{pmatrix} psi$$

Results of Laboratory Unconfined Compression Strength Tests

$\mu_{UCRock} := \text{mean}(UCRock)$ $\mu_{UCRock} = 4550\,psi$ Mean Strength of sample

$\sigma_{UCRock} := \text{Stdev}(UCRock)$ $\sigma_{UCRock} = 1445\,psi$ Standard Deviation of sample

Approximately 90% confidence that strength of intact rock is greater than:

$StrengthIntRock := \mu_{UCRock} - 1.28 \cdot \sigma_{UCRock}$ $StrengthIntRock = 2700\,psi$

Multiply by 10 percent :

$UltStrRockGrout := \text{if}(StrengthIntRock \cdot 0.10 < 450psi, StrengthIntRock \cdot 0.10, 450psi)$

$\boxed{UltStrRockGrout = 270\,psi}$

Allowable Rock-Grout Bond:

$\tau_{a\_calc} := \dfrac{UltStrRockGrout}{3}$ $\tau_{a\_calc} = 90\,psi$

Allowable/working bond stress (includes FS = 3) as recommended by Wyllie and by GEC-4

τ_a = Allowable bond strength = $\sigma u/30$, where σu is the uniaxial compressive strength of the rock (Wyllie, Eq 9.9).
From laboratory results: τa = 100 to 200 psi
From Wyllie, Table 9.2 for Medium rock: τ_a = 100 to 150 psi
From AASHTO, Table 4.4.8.1.2B, for Schist: τa = 45 to 700 psi
Use design bond strength of 45 psi

Use design bond of:

$\boxed{\tau_a := 45psi}$

9.3.2 Wyllie

$l_b := \dfrac{Q}{\pi\, d \cdot \tau_a}$ Where l_b is the bond length and Q is the applied tensile load, Wyllie, Eq 9.8

$$d = \begin{pmatrix} 3 \\ 4 \\ 5 \\ 6 \end{pmatrix} in \qquad l_b = \begin{pmatrix} 6.9 \\ 5.2 \\ 4.1 \\ 3.4 \end{pmatrix} ft$$

Wyllie recommends limits of 6 inch maximum drill hole diameter in rock. Practical limit on length of bond zone is 26 to 33 ft.

Steel-grout stress:

$FOS := 2$

yield strength of steel in MPa

$\sigma_y := 160\,ksi$ $\sigma_y = 1.1 \times 10^3\,MPa$ $\sigma_y := 1102$

comp. strength of grout in MPa

$\sigma_{uc} := 4000\,psi$ $\sigma_{uc} = 27.6\,MPa$ $\sigma_{uc} := 27.6$

For 35mm (1 3/8 in) bars and smaller

$A_b := \pi \cdot \frac{(35mm)^2}{4}$ $A_b = 962\,mm^2$ $A_b := 962$ mm^2

$$I_{d35} := \frac{0.019 \cdot A_b \cdot \sigma_y}{\sqrt{\sigma_{uc}} \cdot FOS}$$

Id is the development length

$I_{d35} = 1.917 \times 10^3$ mm

= 6.3 **ft**

APPENDIX H

Representative Photographs

Photo No. 1 - Collapsed Wall Section Around CMP Pipe Culvert

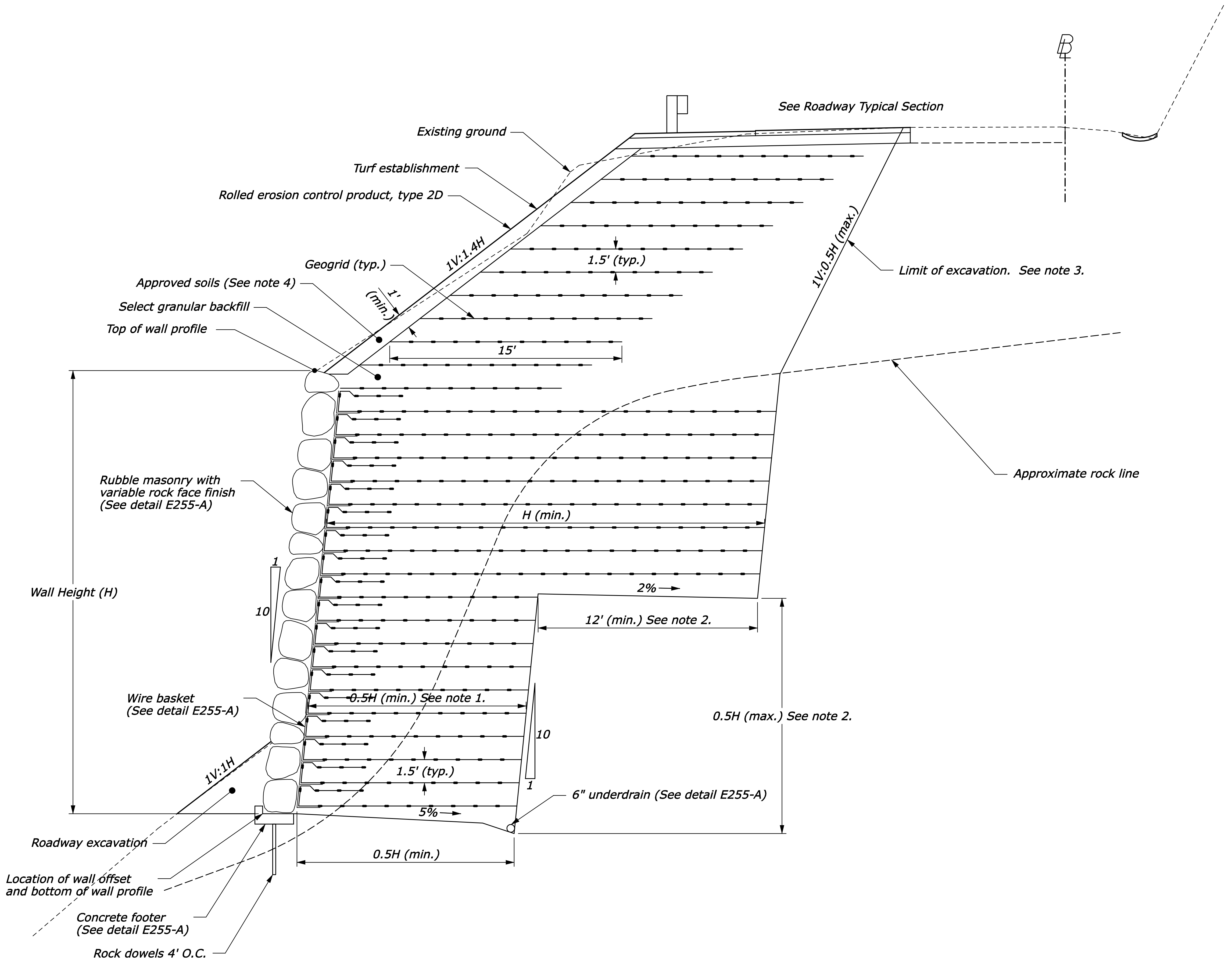
Photo No. 2 – Southern Bulging Wall Section

Photo No. 3 – Looking South, Mortared Wall Face and Rockfill

Photo No. 4 – Northern End of Wall Slide

APPENDIX I
Typical Cross Sections

NPS NO.	REG	STATE	PROJECT	SHEET NO.
601 43953	NE	NC	PRA-BRLI 2P16 SLIDE	B1



Not to Scale

Note:

1. If rock is not encountered, continue to excavate to a maximum geogrid embedment depth of 0.7H.
2. Bench height and width may vary to accommodate construction equipment.
3. Follow OSHA safety regulations for sloping and stabilizing the sides of excavation.
4. Conserve existing soil for reuse .

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FEDERAL HIGHWAY ADMINISTRATION
EASTERN FEDERAL LANDS HIGHWAY DIVISION
STERLING, VIRGINIA

BLUE RIDGE PARKWAY

TYPICAL SECTION WALL RECONSTRUCTION

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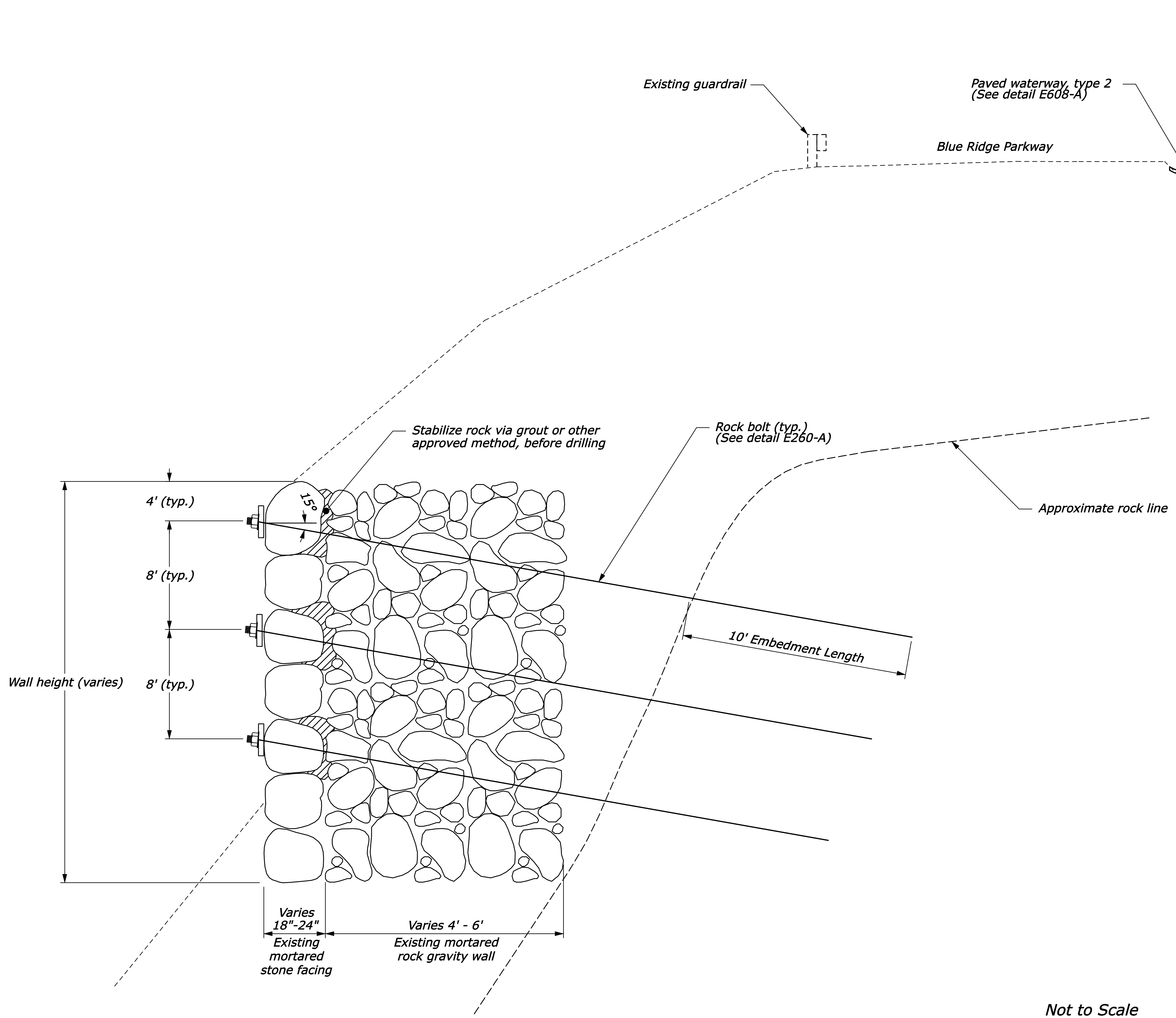
NPS NO.	REG	STATE	PROJECT	SHEET NO.
601 43953	NE	NC	PRA-BRLI 2P16 SLIDE	B3



Note:

1. Vary the location of rock bolts to place in the center of selected large stones, as directed by the CO.

2. Begin stabilization grouting for rock bolts at the bottom row of rock bolts. Proceed to the next row after grout has hardened.

Not to Scale

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STERLING, VIRGINIA

BLUE RIDGE PARKWAY

TYPICAL SECTION WALL STABILIZATION

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